

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PELICAN RIVER BRIDGE REPLACEMENT
HIGHWAY 72, TOWN OF SIOUX LOOKOUT, ONTARIO
W.P. 6940-10-01, SITE #41S-9**

Geocres Number: 52J-10

Report to

MMM GROUP LIMITED

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

October 30, 2014
File: 19-1351-197

H:\19\1351\197 NWR 32 Rehabs\Reports & Memos\
Pelican River Bridge\Pelican River Bridge FIDR_FINAL.doc

TABLE OF CONTENTS

PART 1: FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING.....	2
4	LABORATORY TESTING	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Asphalt	3
5.2	Fill	4
5.3	Peat.....	4
5.4	Silty Clay	4
5.5	Silt to Sandy Silt	5
5.6	Silty Sand to Sand.....	6
5.7	Sand and Gravel	6
5.8	Bedrock	7
5.9	Water Levels	8
6	MISCELLANEOUS	9

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	GENERAL.....	10
8	STRUCTURE FOUNDATIONS.....	10
8.1	Spread Footings on Native Soil or Engineered Fill	11
8.2	Driven H-Pile Foundations	11
8.2.1	Axial Resistance	11
8.2.2	Pile Tips.....	12
8.2.3	Pile Installation	12
8.2.4	Pile Lateral Resistance.....	12
8.3	Downdrag.....	14
8.4	Integral Abutment Considerations	14
8.5	Caissons / Drilled Shafts	15
8.6	Recommended Foundation	15
8.7	Frost Cover.....	15
8.8	Impact on Existing Foundations	15
9	EXCAVATION AND DEWATERING	16
10	SHEET PILE WALLS	16
11	APPROACH EMBANKMENTS	17
12	SCOUR AND EROSION PROTECTION	19
13	LATERAL EARTH PRESSURES	19

14	SEISMIC CONSIDERATIONS	20
15	CONSTRUCTION CONCERNS	21
16	CLOSURE	22

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Borehole Logs and Location Plan from Previous Investigation
Appendix D	Site Photographs
Appendix E	Foundation Comparison
Appendix F	List of Standard Specifications and Special Provisions
Appendix G	Slope Stability Analysis
Appendix H	Borehole Locations and Soil Strata Drawings

FOUNDATION INVESTIGATION AND DESIGN REPORT
PELICAN RIVER BRIDGE REPLACEMENT
HIGHWAY 72, TOWN OF SIOUX LOOKOUT, ONTARIO
W.P. 6940-10-01, SITE #41S-9

Geocres Number: 52J-10

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for the proposed replacement of the Pelican River Bridge on Highway 72, located in the Town of Sioux Lookout, at the crossing of the Pelican River.

The purpose of the investigation was to explore the subsurface conditions at the site, and based on the data obtained, to provide a borehole location plan, record of borehole sheets, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

A previous foundation investigation was carried out at this site prior to construction of the existing bridge (Soil Conditions and Foundations, Proposed Pelican River Bridge Replacement, Highway 72, Sioux Lookout, Ontario; prepared by H. Q. Golder and Associates Limited; dated May 1965; Geocres No. 52J-4). The information presented in the above report was reviewed and incorporated in the current investigation, and the borehole logs and location plan are provided in Appendix C for information purposes.

2 SITE DESCRIPTION

The bridge site is located on Highway 72 approximately 300 m south of Wellington St. in the Town of Sioux Lookout, Ontario. The Pelican River flows westerly through a swampy area into the Pelican Lake. The bridge site is located approximately 150 m east of the river mouth.

The general area is characteristic of a river flood plain. The land surrounding the site is gently undulating and heavily treed, with occasional low rock outcrops. Photographs of the bridge and surrounding area are presented in Appendix D.

The site lies within the physiographical area of Canadian Shield, which is characterized by Pre-Cambrian igneous and metamorphic bedrock typically occurring as rounded knobs and ridges where exposed. According to Canadian Geological Survey (CGS) data, the bedrock at this site generally

consists of mafic to intermediate meta-volcanic rocks of the Winnipeg River Subprovince. The bedrock is overlain by a discontinuous cover of Pleistocene sands and gravels (glaciofluvial outwash) and silts and sands (glaciofluvial outwash) overlain by silt and clay (glaciolacustrine deposit).

3 SITE INVESTIGATION AND FIELD TESTING

The previous foundation investigation for the existing bridge consisted of advancing a total of twelve test holes including seven sampled boreholes (Boreholes 1 to 7) and five dynamic cone penetration test (DCPT) holes (Boreholes 8 to 12). Three boreholes (Boreholes 4, 6 and 7) were drilled on land and the remaining four were advanced from the river ice surface. Standpipe piezometers were installed in Boreholes 3 and 5. Bedrock was encountered in Borehole 3 and proved by coring 3 m.

The current site investigation and field testing for this project were carried out during the period of May 30 to June 4, 2014. A total of six boreholes, identified as PRB-01 to PRB-06, were advanced to depths ranging from 9.8 to 19.9 m below the ground surface. Details of the borehole locations, drilling depths and completion details are summarized in Table 3.1 below.

Table 3.1 – Borehole Summary

Location	Boreholes	Drilling/Coring Depth (m)	Completion Details
South Approach	PRB-01	9.8	Borehole backfilled with bentonite holeplug to 2.1 m, then cuttings to ground surface.
South Abutment	PRB-02	17.9	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen installed. Bottom of screen located at 13.7 m, sand filter to 11.6 m and bentonite holeplug to 10.8 m, bentonite holeplug mixed with cuttings to 0.6 m and then cuttings to ground surface.
	PRB-03	19.9	Borehole backfilled with bentonite holeplug mixed with cuttings to 0.3 m, then concrete to 0.1 m, and asphalt to ground surface.
North Abutment	PRB-04	18.5	Borehole backfilled with bentonite holeplug mixed with cuttings to 0.2 m, then concrete to 0.1 m, and asphalt to ground surface.
	PRB-05	17.2	Standpipe piezometer consisting of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen installed. Bottom of screen located at 16.8 m, sand filter to 14.9 m and bentonite holeplug to 14.1 m, bentonite holeplug mixed with cuttings to 0.6 m and then cuttings to ground surface.
North Approach	PRB-06	9.8	Borehole backfilled with bentonite holeplug mixed with cuttings to ground surface.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawings included in Appendix H.

All boreholes were advanced using a CME55 track-mounted drill rig in combination with hollow stem augers and NW casing/tri-cone methods to advance the boreholes in the overburden. Samples of the overburden soils were obtained from the boreholes at selected intervals using a split spoon

sampler in conjunction with Standard Penetration Testing (SPT).

Core samples of the underlying bedrock were recovered from selected boreholes using NQ rock coring equipment. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed during the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed in boreholes PRB-02 and PRB-05. Following the final water level reading, the piezometers were decommissioned in general accordance with MOE Regulation 903.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets included in Appendix A. Selected samples were also subjected to gradation analysis and the results of this testing program are summarized on the Record of Borehole sheets in Appendix A and shown on the figures included in Appendix B.

Point load tests (PLT) were performed on selected intact rock core samples. Unconfined compressive strengths (UCS) of the rock cores correlated from the PLT results are shown on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings in Appendix H. Boreholes 1 to 7 from the previous investigation (referenced hereafter as previous boreholes) were considered in the current investigation. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

The subsurface stratigraphy below the existing embankment fill encountered at the site generally consists of a surficial layer of amorphous peat overlying a layer of varved silty clay transitioning to clayey silt to silt which is underlain by glaciofluvial deposits consisting of sand and gravel to silty sand. Bedrock was encountered beneath the sand to silty sand deposit at the south abutment. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt pavement was encountered in Boreholes PRB-03 and PRB-04. The thickness of the asphalt ranged from 80 to 100 mm in the boreholes.

5.2 Fill

Existing embankment fill was encountered below the asphalt pavement in Boreholes PRB-03 and PRB-04 and immediately at the ground surface in Boreholes PRB-01, PRB-02, PRB-05 and PRB-06. The brown fill is heterogeneously composed of sand and gravel, clayey silt and silty sand. The thickness of the fill ranged from 2.8 to 5.6 m, with the base of the fill at Elev. 354.7 to 357.4.

SPT 'N' values recorded in the fill ranged from 2 to 31 blows per 0.3 m penetration, indicating a very loose to dense relative density. An SPT 'N' value of 50 blows for 0.1 m penetration was recorded in Borehole PRB-03 at 1.5 m depth, indicating presence of cobbles. Moisture contents of the fill ranged from 1 to 23% with typical values between 8 and 20%.

The results of grain size analyses conducted on fill samples are provided on the Record of Borehole sheets in Appendix A and illustrated in Figures B1a through B1c of Appendix B. The results are summarized as follows:

	Sand & Gravel	Clayey Silt	Silty Sand
Gravel	6 to 41%	5%	17%
Sand	50 to 91%	27%	59%
Silt & Clay	3 to 9%	-	-
Silt	-	42%	21%
Clay	-	26%	3%

5.3 Peat

A surficial peat layer was encountered below ice/water in all the previous boreholes, and below the existing fill in all boreholes except PRB-02 in the current study. The black to dark brown peat was described as amorphous with fine root structure. The thickness of the peat ranged from 1.4 to 2.1 m in the previous boreholes and from 0.7 to 1.2 m in the current boreholes, with the base of the peat at Elev. 354.9 to 355.9 in the previous boreholes and at Elev. 354.7 to 356.6 in the current boreholes.

In the previous boreholes, SPT 'N' values of zero (Weight of Rod to Weight of Hammer) to 3 blows per 0.3 m penetration were recorded in the peat. Field vane shear tests (VST) carried out in the peat measured undrained shear strengths ranging from 5 to 25 kPa, indicating very soft to soft consistency.

SPT 'N' values recorded in the peat in the current boreholes ranged from 3 to 9, indicating soft to stiff consistency. The increased penetration resistance is likely resulted from consolidation of the peat under the weight of the existing embankment fill.

Natural moisture contents of the peat ranged from 135 to 585% in the previous boreholes and from 100 to 313% in the current boreholes.

5.4 Silty Clay

A layer of grey silty clay was encountered below the fill in Borehole PRB-02 and below the

peat in all other boreholes. Where fully penetrated, the thickness of the layer ranged from 2.2 to 5.5 m with the lower boundary encountered at Elev. 350.0 to 354.4.

A thin layer of organic silt was encountered at the top of the silty clay layer in Boreholes 1, 2, 5 and 6. The thickness of the organic silt ranged from 0.5 to 0.8 m. Measured organic contents of the organic silt samples ranged from 20 to 35%. The undrained shear strength of this organic silt layer was reported to range from 10 to 12 kPa.

SPT 'N' values recorded in the silty clay in the previous boreholes were generally zero blows per 0.3 m penetration (Weight of Rod to Weight of Hammer). Field vane shear tests (VST) measured undrained shear strengths ranging from 5 to 20 kPa. Based on the SPT and VST data, the consistency of the silty clay varied from very soft to soft.

SPT 'N' values recorded in the silty clay in the current boreholes ranged from zero to 4 blows per 0.3 m penetration. Field vane shear tests (VST) measured undrained shear strengths ranging from 17 to 48 kPa, generally increasing with depth. Based on the SPT and VST data, the consistency of the silty clay varies from soft to firm. Sensitivity of the silty clay, calculated as a ratio of undisturbed strength to remoulded strength, ranged from 3 to 8, suggesting that the silty clay is low to medium sensitive.

The results of grain size analyses conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B2 of Appendix B. The results are summarized as follows:

Gravel	0%
Sand	0 to 1%
Silt	33 to 54%
Clay	46 to 67%

The results of Atterberg Limits tests conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A and illustrated in Figure B5 of Appendix B. The results indicated that the deposit has plastic limits ranging from 17 to 24% and liquid limits ranging from 29 to 63%, suggesting low to high plasticity. Plasticity indices, the difference between the plastic limit and liquid limit, ranged from 11 to 38%.

Natural moisture contents of the organic silt ranged from 102 to 244% in the previous boreholes. Natural moisture content of the inorganic silty clay ranged from 27 to 95% in the previous boreholes and 35 to 81% in the current boreholes.

5.5 Silt to Sandy Silt

A layer of grey silt to sandy silt was encountered below the silty clay in all the boreholes. The silt to sandy silt contains trace to some clay. Where fully penetrated, the thickness of the silt to sandy silt layer ranged from 1.4 to 3.5 m, and the lower boundary was encountered at Elev. 347.4 to 351.6. PRB-06 was terminated in the silt at a depth of 9.8 m or Elev. 350.4.

SPT 'N' values recorded in the silt to sandy silt layer ranged from 0 to 28 blows per 0.3 m

penetration, indicating a very loose to compact relative density. Natural moisture contents of the silt to sandy silt ranged from 19 to 31%.

The results of grain size analyses conducted on silt samples are provided on the Record of Borehole sheets in Appendix A, and plotted in Figure B3 of Appendix B. The results are summarized as follows:

Gravel	0%
Sand	0 to 27%
Silt	66 to 85%
Clay	5 to 15%

5.6 Silty Sand to Sand

A layer of grey silty sand to sand, some gravel was encountered below the silt to sandy silt deposit in Boreholes PRB-01 to PRB-05. The silty sand contains occasional cobbles and boulders. The layer was fully penetrated in PRB-02 and PRB-03 where a thickness of 4.7 m and 5.5 m was recorded, and the lower boundary was encountered at 14.9 m and 17.4 m depth (Elev. 345.4 and 342.9). PRB-01, PRB-04 and PRB-05 were terminated within the silty sand at Elev. 342.0 to 350.4.

SPT 'N' values recorded in the silty sand to sand ranged from 8 blows per 0.3 m penetration to 100 blows for 0.075 m penetration, indicating a loose to very dense relative density. Measured natural moisture contents ranged from 5 to 15%.

The results of grain size analyses conducted on samples of the silty sand to sand deposit are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B4 of Appendix B. The results are summarized as follows:

Gravel	2 to 17%
Sand	60 to 77%
Silt & Clay	6 to 30%

5.7 Sand and Gravel

A layer of sand and gravel was encountered below the silt to sandy silt layer in Boreholes 1 to 7. The sand and gravel contains occasional cobbles and boulders. The top of this layer starts at about Elev. 348 to 350 and a number of the boreholes were terminated in this layer at Elev. 340.9 to 345.3. The layer thickness ranged from 0.2 m to greater than 7 m.

SPT 'N' values recorded in the deposit typically ranged from 36 to greater than 100 blows per 0.3 m penetration, indicating a dense to very dense relative density. Measured natural moisture contents ranged from 5 to 15%.

5.8 Bedrock

Porphyritic bedrock was encountered in Borehole 3 below the sand and gravel and in PRB-02 and PRB-03 below the silty sand. Table 5.1 summarizes the depth to bedrock and the bedrock surface elevations determined in the boreholes.

Table 5.1: Depth to Bedrock at Borehole Locations

Foundation Element	Borehole	Depth to Bedrock (m)	Bedrock Elevation (m)
South Abutment	3	9.6	347.7
	PRB-02	14.9	345.4
	PRB-03	17.4	342.9

The bedrock was only encountered in the boreholes drilled near the south abutment. Bedrock was not encountered within the depths of the boreholes drilled at the piers and north abutment. The stratigraphic profile drawing in Appendix H indicates sloping bedrock at the south abutment.

The bedrock is generally described as slightly weathered to fresh, dark grey with light grey quartz inclusions. Total Core Recovery (TCR) in the bedrock ranged from 89 to 100%. The Rock Quality Designation (RQD) determined from the recovered cores generally ranged from 67 to 100%, indicating fair to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 3.

The unconfined compressive strength (UCS) of the rock, estimated from the results of point load tests conducted on the rock core samples, ranges between 149 and 307 MPa, indicating a very strong to extremely strong rock. The point load test results are included on the borehole logs in Appendix A.

5.9 Water Levels

The water levels in the boreholes were measured upon completion. However, water was used during the wash-boring and coring operations and therefore the measured water levels may not reflect prevailing groundwater levels at the site. Standpipe piezometers were installed in Boreholes PRB-02 and PRB-05 to monitoring groundwater levels after drilling. The water levels measured in the open boreholes upon completion of drilling and in the piezometers are summarized in Table 5.2. Historical water level measurements in standpipe piezometers installed in the previous Boreholes 3 and 5 are also included.

Table 5.2: Water Level Measurements

Borehole	Date	Water Level (m)		Remark
		Depth	Elevation	
3	Jan. 14, 1965	0.2	357.1	In piezometer
5	Jan. 12, 1965	0.1	357.1	In piezometer
PRB-01	May 31, 2014	3.8	356.4	In open borehole
PRB-02	May 31, 2014	1.3	359.0	In piezometer
	June 02, 2014	1.2	359.1	
	June 04, 2014	1.2	359.1	
PRB-04	June 03, 2014	1.9	358.6	In open borehole
PRB-05	June 03, 2014	0.7	359.8	In piezometer
	June 04, 2014	0.6	359.9	
PRB-06	May 31, 2014	1.8	358.4	In open borehole

The approximate 2-year high water level in the river shown on the preliminary GA drawing is at Elev. 357.7 m. In May 2014, the river level was assessed to be at approximate elevation of 358.7. The groundwater levels measured in the piezometers in the current boreholes are up to 2.2 m above the high water level in the river. The river and groundwater levels are expected to fluctuate seasonally and subject to precipitation patterns, and may vary from the levels presented above.

6 MISCELLANEOUS

Eastern Ontario Diamond Drilling Ltd. of Hawkesbury, Ontario supplied the drill rig and conducted the drilling, sampling and in-situ testing operations. A track-mounted CME #55 drill rig was used for the duration of the investigation.

The drilling and sampling operations were supervised in the field by Mr. Stephane Loranger of Thurber. Mr. Mark Farrant, P.Eng. directed the field operations.

The report was prepared by Mr. Keli Shi, P.Eng., and reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects.

THURBER ENGINEERING LTD.

Keli Shi, P.Eng., M.Eng.
Geotechnical Engineer



Murray R. Anderson, P.Eng., M.Eng.
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng., Ph.D.
Review Principal



FOUNDATION INVESTIGATION AND DESIGN REPORT
PELICAN RIVER BRIDGE REPLACEMENT
HIGHWAY 72, TOWN OF SIOUX LOOKOUT, ONTARIO
W.P. 6940-10-01, SITE #41S-9

Geocres Number: 52J-10

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed replacement bridge.

At present, Highway 72 crosses the Pelican River on a fifteen (15) span structure with a total length of 71.6 m, supported on timber piles driven to compact to very dense sand and gravel. The existing road grade on the bridge is at approximate Elev. 360.3 to 360.6 m.

The preliminary General Arrangement drawing indicates that the replacement bridge will be a four span structure with a total length of 66 m. Finished road grade will be essentially unchanged. Sheet pile walls are proposed at both abutments to retain the approach fill.

The discussion and recommendations presented in this report are based on information provided by MMM Group Limited, on the soil conditions presented in the Geocres report and on the factual data obtained in the course of this investigation.

8 STRUCTURE FOUNDATIONS

In general, the soil stratigraphy below the existing embankment fill consists of a surficial peat layer underlain by a very soft to firm silty clay deposit which overlies a relatively thin layer of loose silt. The silt is underlain by glaciofluvial outwash materials varying from silty sand to sand and gravel. Bedrock was encountered in the south abutment area only, at depths of 9.6 to 17.4 m (Elev. 347.7 to 342.9). The river level was estimated to rise to Elev. 358.7 in May 2014. Groundwater levels measured in the previous investigation were at Elev. 357.1, which was at or slightly below the ice surface at the time of investigation. Groundwater levels measured in the current investigation ranged approximately from Elev. 359 to 360.

Based on the subsurface conditions, initial consideration was given to supporting the replacement bridge on spread footings on native soil or engineered fill, driven steel H-piles and augered caissons. A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix E.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme from a geotechnical perspective is recommended.

8.1 Spread Footings on Native Soil or Engineered Fill

The use of spread footings to support the abutments is not recommended given the relatively low geotechnical resistance available in the native soils and the potential for large consolidation settlement in the peat and silty clay deposits. Construction of engineered fill to support the footings is not feasible in view of the depth of peat and silty clay to be excavated and the high water level.

Similarly, supporting the piers on spread footings on the river bottom is not feasible due to the low geotechnical resistance available in the underlying peat and silty clay, and the need for deep sheet pile wall cofferdams to enable excavation and dewatering within the river. Footings will also have to be protected from scour and erosion.

8.2 Driven H-Pile Foundations

8.2.1 Axial Resistance

The ground conditions at the site are considered to be suitable for the use of steel H-piles driven to refusal on bedrock at the south abutment, in very dense sand and gravel at the piers, and in very dense silty sand at the north abutment.

We understand that the structural designers propose to use an HP360x132 pile section for the foundations at this site. HP360x132 piles founded in the strata noted above should be designed using the recommended geotechnical resistances presented in Table 8.1. The SLS reaction will not govern for piles driven to bedrock.

Table 8.1 – Recommended Geotechnical Resistance for HP360x132

Foundation Element	Founding Stratum	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
South Abutment	Bedrock	2,400	Does not govern.
Piers 1, 2, and 3	Sand & Gravel	1,550	1,200
North Abutment	Silty Sand	1,400	1,200

The estimated tip elevations of piles driven to the bedrock surface, are presented in Table 8.2.

Table 8.2 – Estimated Pile Tip Elevation for HP360x132

Foundation Element	Borehole	Estimated Pile Tip Elevation (m)
South Abutment	PRB-02	345.4
	PRB-03	342.9
Pier 1	8	342.0 *
Pier 2	1, 2	339.5 *
Pier 3	5	340.7 *
North Abutment	PRB-04, PRB-05	344.8 *

Note: * Actual refusal elevation for each pile may vary during installation.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any fills through which the piles will be driven.

8.2.2 Pile Tips

Pile tip protection is recommended for driven H-piles to prevent pile damage when setting the piles on bedrock or if cobbles or boulders are encountered. The tips of all driven H-piles at the piers and north abutment must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. The use of rock points (Titus Rock Injector or equivalent) is recommended for driving piles to the sloping bedrock surface at the south abutment.

8.2.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

The alignment of the H-piles should be carefully selected to avoid the existing timber bents and piles.

The appropriate pile driving note is “Piles to be driven to bedrock” at the south abutment.

Pile driving at the piers and north abutment must be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and an ultimate pile resistance should be specified by the designer. The Hiley formula need not be used until the piles are within 2.0 m of the design pile tip elevation. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile. “R” must have a minimum value of twice the design load at ULS, but must not exceed 3,100 kN at the piers and 2,800 kN at the north abutment.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerance, a driving template or other means may be required to achieve the specified maximum deviation.

8.2.4 Pile Lateral Resistance

The geotechnical lateral resistance acting on a pile in cohesionless soils may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \gamma' z K_p \quad (\text{kPa})$$

Where z = depth of embedment of pile (m)

D = pile width or diameter (m)

n_h = coefficient related to soil relative density (kN/m^3)

γ' = effective unit weight (kN/m^3)

K_p = passive earth pressure coefficient

The geotechnical lateral resistance acting on a pile in cohesive soils may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = 67 S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 S_u \quad (\text{kPa})$$

Where S_u = undrained shear strength (kPa)

D = pile width or diameter in metres

The above equations and recommended parameters in Table 8.3 below may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

Table 8.3 – Soil Parameters for Lateral Pile Resistance

Soil Unit	Elevation (m)		γ' (kN/m ³)	n_h (kN/m ³)	K_p	S_u (kPa)
	Top	Bottom				
South Abutment						
Fill	358.5 *	355.9	10	2,500	3.0	-
Peat	355.9	354.7	3	-	-	10
Silty Clay	354.7	351.5	8	-	-	20
Silt	351.5	348.4	10	1,500	2.8	-
Silty Sand to Sand (Compact to Dense)	348.4	Bedrock	11	3,500	3.3	-
Piers						
Peat	358.5 to 357.5 *	354.9	3	-	-	10
Silty Clay	354.9	350.8	8	-	-	7.5
Silt	350.8	347.8	10	1,500	2.8	-
Sand & Gravel (Compact to Dense)	347.8	346.0	11	3,000	3.3	-
Sand & Gravel (Very Dense)	346.0	344.0 **	13	10,000	4.2	-
North Abutment						
Fill	358.5 *	356.5	10	2,500	3.0	-
Peat	356.5	355.3	3	-		10
Silty Clay	355.3	351.5	8	-	-	20
Silt	351.5	348.7	10	1,500	2.8	-
Silty Sand (Compact)	348.7	347.0	11	3,500	3.3	-
Silty Sand (Very Dense)	347.0	344.8 **	12	6,500	3.7	-

Note: * Assumed finished ground surface at pile locations.

** Pile tip elevations vary at pile locations.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate

lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} L D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.4. Intermediate values may be obtained by linear interpolation.

Table 8.4 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing, Centre to Centre	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

In the case of conventional abutments, i.e. not integral type, horizontal loads may be resisted by means of battered piles. Additional lateral resistance could also be provided by socketing the piles into bedrock. However, considering the depth to bedrock at this site, socketing of the piles is not expected to be an efficient means of developing lateral resistance.

8.3 Downdrag

The site is underlain by compressible peat and silty clay deposits. Placement of additional approach fill behind the abutment walls or to raise grades will result in development of downdrag forces along the length of the abutment piles due to consolidation of the peat and clay under the weight of the new fill.

We understand however that the proposed design will incorporate expanded polystyrene (EPS) as backfill behind the sheet pile abutments to eliminate any load increase and prevent post-construction settlement of the approach fill. In addition, no grade raise is planned. As no new net load will be applied to the foundation soils, downdrag on the piles is not an issue.

8.4 Integral Abutment Considerations

The use of H-piles at the abutments allows for the design of an integral abutment structure. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. At this site, the lateral resistance of a pile should provide sufficient flexibility. However, to provide the required flexibility for piles installed through compacted fill, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is installed, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract documents specifying the gradation of the sand according to Table 8.5.

Table 8.5 – Integral Abutment Sand Backfill Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

8.5 Caissons / Drilled Shafts

Augered caissons bearing on the bedrock surface or socketed into bedrock could be considered at the south abutment. However, construction of caissons in the cohesionless soils below groundwater level will require the use of a permanent liner, and sealing of the caisson liner into the bedrock to prevent inflow of water and sands/silts may be problematic.

In view of the groundwater level and presence of cohesionless deposits over the bedrock, the use of caissons is not recommended and has not been developed herein.

8.6 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, steel H-piles driven to refusal on bedrock at the south abutment or to the design resistance in very dense cohesionless soils at the piers and north abutment are the preferred foundation option at this site.

8.7 Frost Cover

The depth of frost penetration at this site is approximately 2.6 m. The base of pile caps must be provided with a minimum of 2.6 m of earth cover as protection against frost action.

8.8 Impact on Existing Foundations

Piles will be driven adjacent to the existing bridge for construction of the replacement bridge.

Archive design drawings indicate that the timber piles supporting the existing bridge were driven approximately 1.0 to 1.5 m into dense to very dense cohesionless deposits. Provided these piles were driven to the dense to very dense materials as designed, driving of the new piles is not likely to impact the performance of the existing pile foundations. However, driving records or other documentation was not available to confirm the pile tip elevations. Therefore it is recommended that the structural designer select appropriate points on the existing structure and specify a monitoring program for the duration of pile driving (including establishment of adequate benchmarks outside the zone of potential influence and acquirement of baseline readings in advance of pile driving).

The Contractor should be prepared to maintain the grade of the existing bridge in operation by such means as lifting and shimming of the structure if necessary. Overdriving of the piles causing disturbance of the overburden soils must not be permitted, particularly on sloping bedrock at the south abutment.

9 EXCAVATION AND DEWATERING

Excavation for bridge replacement and EPS fill placement behind the abutments is expected to extend to approximate Elev. 358.0 and will be limited to the existing approach fill. The approximate 2-year high water level in the river is Elev. 357.7 m, and the high water level in May 2014 was estimated at Elev. 358.7. Provided the work is not carried out during a period of unusually high water levels, the excavation is not expected to extend below the water level.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill may be classified as Type 3 soil above the water table and as Type 4 soil below the water table. Flatter slopes may be required at locations where water seepage affects surficial stability.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

The selection of the method of excavation is the responsibility of the Contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, and cobbles.

Roadway protection will be required to facilitate staged construction at this site. The temporary excavation support system should be designed and constructed in accordance with OPSS 539. Sheet piles or soldier pile and lagging walls are considered appropriate for roadway protection. The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes.

10 SHEET PILE WALLS

The current design proposes the installation of steel sheet pile walls adjacent to the pile foundations in lieu of conventional abutment walls. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill. The alignment of the proposed sheet pile walls should be carefully selected to avoid existing timber bents and piles.

Lateral stability of the sheet pile walls should be checked by the wall designer using the parameters presented in Table 10.1. The coefficients of passive earth pressure (K_p) are provided for horizontal ground surface in front of the sheet pile wall. For sloping ground in front of the sheet pile wall, the recommended values for the coefficients of passive earth pressure (K_p) should be reduced. The possibility of material loss due to river erosion in front of the sheet piles should also be considered in the check of the lateral earth pressure balance.

Table 10.1 – Soil Parameters for Sheet Pile Analysis

Foundation Element	Soil Unit	Elevation (m)		γ' (kN/m ³)	K_a	K_p
		Top	Bottom			
South Abutment	Fill	360.3 *	355.9	20	0.33	3.0
	Peat	355.9	354.7	3	0.35	- **
	Silty Clay	354.7	351.5	8	0.38	2.6
	Silt	351.5	348.0	10	0.36	2.8
North Abutment	Fill	360.5 *	356.5	20	0.33	3.0
	Peat	356.5	355.3	3	0.35	- **
	Silty Clay	355.3	351.5	8	0.38	2.6
	Silt	351.5	348.0	10	0.36	2.8

* Top of sheet pile elevation varies.

** Passive resistance in the peat is ignored.

In general, backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or Granular B Type III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501. It is understood that at this site, EPS will be placed behind the abutment sheet piles to apply no load increase on the compressible foundation soils, in order to mitigate settlement of the approaches.

Driving of the sheet pile through the existing approach fill may encounter cobbles. Removal of any such obstructions may be required to install the sheeting. Any visible obstructions such as boulders and rock protection along the sides of the embankment should be removed prior to driving the sheet piles. Tip protection is recommended for the sheet piles.

In light of the soft foundation clay and the underlying sensitive silt deposit, vibratory methods must not be used at this site to install sheet piles.

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The native soils in front of the sheet piles should be protected from river erosion so that the sheet piles do not lose lateral support.

11 APPROACH EMBANKMENTS

No grade raise is proposed at the approach embankments. However, additional fill placement is required behind the new abutments as approach embankments will be extended towards the river by approximately 3 m at both abutments. The foundation soils governing stability of the approach embankments consist of compressed peat and very soft to firm silty clay.

If conventional granular materials are used, settlement induced by the additional embankment fill is estimated to be in the order of 300 mm at both abutments. We understand that the use of lightweight fill material (expanded polystyrene (EPS) blocks) is planned to eliminate the load increase on the compressible foundation soils and reduce the settlement to negligible levels along the approach

embankment between the existing abutment and the new abutment. An NSSP addressing supply and installation of EPS fill should be included in the contract documents.

Global stability analyses were carried out to assess the stability of the river facing slopes under the existing conditions and with the proposed sheet pile wall configuration. The stability analyses were carried out using the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. The geotechnical model and results of the analyses are shown on Figures 1 to 4 in Appendix G. The computed factors of safety are summarized in Table 11.1.

Table 11.1 - Computed Factors of Safety for Approach Embankments

Abutment	Condition	Sheet Piles	Factor of Safety	Figure (Appendix G)
South	Short term - undrained	No	1.28	1
	Long term - drained	Yes	2.87	2
North	Short term - undrained	No	1.19	3
	Long term - drained	Yes	2.27	4

According to the Geocres report, stability of the river bank slopes was a concern during the design and construction of the existing bridge. Stabilizing berms were constructed then to improve the global stability of the approach embankments. Analyses for the existing approach embankments without sheet piles indicated factors of safety ranging from 1.2 to 1.3 for undrained conditions, and greater than 1.5 for drained conditions, which are representative of the current ground conditions. Stability analyses carried out for the sheet-pile retained approach embankments using EPS backfill indicated that the sheet piles with tips driven to Elev. 353.0 (approximately 2 m into the silty clay) at the proposed abutments will increase the factors of safety to above 2.0 under long term conditions.

The depth of sheet piles will also be governed by temporary construction conditions such as a heavy crane loading on the approach embankments during pile driving or girder lifting. A preliminary analysis of a typical crane loading indicates that the sheet piles should be driven deeper, to a tip elevation of 348.0 (a depth of penetration of approximately 10.5 m) to maintain stability of the approaches under the temporary crane loading.

Based on these analyses, it is recommended that a minimum depth of sheet pile penetration of 10.5 m (Elev. 348.0) be adopted. The depth of penetration may need to be greater to provide lateral stability.

Design of the lightweight fill embankment at this site must consider hydrostatic uplift. For design purposes, a factor of safety against hydrostatic uplift of 2.0 is recommended at the highest river level anticipated during the design life of the bridge.

Embankment construction should be in accordance with OPSS.PROV 206. It is recommended that embankment fill in addition to the EPS consist of granular materials. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501. The backfill to the abutment walls should be in accordance with OPSS 902.

12 SCOUR AND EROSION PROTECTION

Erosion protection should be provided along any soil surfaces that may be in contact with the river flow. In particular, erosion protection must be provided in front of the sheet pile walls to prevent undermining of the sheet pile walls at the abutments.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

13 LATERAL EARTH PRESSURES

Earth pressures acting on the structure may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K (\gamma h + q)$$

where: p_h = horizontal pressure on the wall at depth h (kPa)

K = coefficient of lateral earth pressure (see Table 13.1)

γ = unit weight of retained soil (see Table 13.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 13.1.

Table 13.1 – Coefficients of Lateral Earth Pressure (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

* For wing walls.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 13.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or III, or at a depth of 1.7 m for Granular A or Granular B Type II.

14 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.00
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.00
- Peak Ground Acceleration 0.036 g

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.5 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading				
Loading Condition	Granular A or Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ$; $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.28	0.42	0.32	0.51
Passive (K_{PE})	3.6	-	3.2	-
At Rest (K_{OE})**	0.47	-	0.52	-

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods (1973).

The loose saturated silt layer underlying the silty clay deposit may be susceptible to liquefaction under seismic loading. However, considering the low seismic activity in the area (acceleration related seismic zone of zero), liquefaction of the foundation soils is not a concern.

15 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Pile driving for the replacement bridge may potentially cause settlement of the existing bridge during staged construction. It is recommended that settlement monitoring of the existing bridge be carried out for the duration of pile driving. The Contractor should be prepared with appropriate equipment on site to maintain the grade of the existing bridge within acceptable tolerance.
- Installation of the sheet piles retaining approach embankments may encounter resistance in the fill due to the presence of cobbles. The Contractor must allow for removal of any such obstructions. Vibratory methods must not be used to install sheet piles at this site.
- The sequence of H-pile and sheet pile installation should be carefully considered to avoid pile alignment problems.
- Piles at the north abutment may encounter refusal in cobbles and boulders present in the dense to very dense silty sand and sand above bedrock. If the pile tip elevations vary by more than 3 m from the predicted values, the design team should be notified and permitted to review the possible implications.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the clay subgrade to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. This is of particular importance due to the presence of the peat layer and soft silty clay stratum at this site. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP addressing this issue is attached in Appendix F.

16 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Keli Shi, P.Eng. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P. K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Keli Shi, P.Eng., M.Eng.
Geotechnical Engineer



Murray R. Anderson, P.Eng., M.Eng.
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng., Ph.D.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS


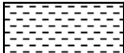



ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	Approximate Uniaxial Compressive Strength (psi)	Field Estimation of Hardness*
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery:(SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation:(RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index:(FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No PRB-01

1 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 889.8 E 382 901.9 ORIGINATED BY SLL
HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2014.05.31 - 2014.05.31 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								<div><div><div>20406080100</div><div></div><div></div></div></div> <div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div><div>+ FIELD VANE</div><div>× LAB VANE</div></div>					
							WATER CONTENT (%)						
							<div><div><div>W P</div><div></div><div>W</div><div></div><div>W L</div></div><div></div></div>						
360.2	GROUND SURFACE												
0.0	Gravelly SAND , trace silt, trace rootlets Brown Moist (FILL)		1	GS			360						22 71 7 (SI+CL)
359.5													
0.7	Clayey SILT , sandy, trace organics Firm Brown Moist (FILL)		1	SS	7		359						
358.8													
1.4	Silty SAND , some gravel, trace clay Loose Brown Wet (FILL)		2	SS	4		358						
357.4			3	SS	3								17 59 21 3
2.8	PEAT , amorphous Firm to Stiff Black Wet						357						
356.6			4	SS	9								
3.6	Silty CLAY , varved, trace rootlets Soft to Firm Grey Moist						356						
			5	SS	0		355						0 0 54 46
354.4													
5.8	SILT , trace clay, trace sand Loose Grey Wet						354						
			6	SS	5								
							353						
			7	SS	5		352						0 6 85 9
351.6													
8.6	Silty SAND , trace gravel Loose Grey Wet						351						
350.4			8	SS	8								
9.8	END OF BOREHOLE AT 9.8m.												

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-01

2 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 889.8 E 382 901.9 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.05.31 - 2014.05.31 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page BOREHOLE OPEN TO 4.2m AND WATER LEVEL AT 3.8m ON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 2.1m, THEN CUTTINGS TO SURFACE.													

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 9/22/14

RECORD OF BOREHOLE No PRB-02

1 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 900.6 E 382 899.5 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.05.30 - 2014.05.31 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
360.3	GROUND SURFACE													
0.0	Gravelly SAND , trace silt Loose Brown Moist to Wet (FILL)		1	SS	7									
358.9														
1.4	Clayey SILT , sandy, trace gravel Soft to Firm Brown Moist (FILL)		2	SS	3									
357.7			3	SS	10									
2.6	Silty SAND , trace gravel Very Loose to Compact Brown Wet (FILL)		4	SS	2									
			5	SS	12									
354.7														
5.6	Silty CLAY , trace roots and organics Firm Grey Moist		6	SS	0									
			7	SS	4									
351.5														
8.8	SILT , trace sand, trace clay Compact Grey Wet		8	SS	10									

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-02

2 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 900.6 E 382 899.5 ORIGINATED BY SLL
HWY 72 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2014.05.30 - 2014.05.31 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								20 40 60 80 100						
	Continued From Previous Page													
350.1														
10.2	Silty SAND , some gravel, trace clay, occasional cobbles Compact to Dense Grey Wet		9	SS	25		350							
							349							
			10	SS	39		348							12 60 24 4
							347							
			11	SS	15		346							
345.4			12	SS	100/		345							
14.9	BEDROCK , porphyry, slightly weathered to fresh, dark grey with light grey inclusions, with some sand infilling joints Horizontal joint at 15.0m, 15.9m Sub-vertical joint (25mm to 50mm) at 15.1m, 15.6m and (200mm) at 16.2m Sub-vertical joint (75mm) at 16.4m, 16.5m and (225mm) at 16.9m With quartz vein at 17.2m		1	RUN	0.075		344							RUN #1 TCR=100% SCR=88% RQD=88% UCS=296MPa (Average)
			2	RUN			343							RUN #2 TCR=100% SCR=100% RQD=67% UCS=307MPa (Average)
342.4	END OF BOREHOLE AT 17.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 31/14 1.3 359.0 Jun 02/14 1.2 359.1 Jun 04/14 1.2 359.1													

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 9/22/14

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-03

1 OF 3

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 904.5 E 382 908.8 ORIGINATED BY SLL
HWY 72 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2014.06.04 - 2014.06.04 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
360.3	GROUND SURFACE							20 40 60 80 100					
0.0	ASPHALT: (100mm) Gravelly SAND , trace silt, occasional cobbles Loose to Dense Brown Moist to Wet (FILL)		1	GS			360						
0.1			1	SS	31		359						25 68 7 (SI+CL)
			2	SS	50/ 0.100		358						
			3	SS	7		357						
			4	SS	31		356						
355.9							355						
4.4		PEAT , amorphous Soft Dark Brown Moist		5	SS	3		354					
354.7	Silty CLAY , trace rootlets Soft to Firm Grey Wet Varved, silt seams		6	SS	0		353						
5.6			7	SS	0		352						
							351						
351.5													
8.8	SILT , some sand, trace clay Loose Grey Wet		8	SS	5							0 16 77 7	

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-03

2 OF 3

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 904.5 E 382 908.8 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.06.04 - 2014.06.04 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W P W W L					
								20 40 60 80 100				20 40 60					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%)					
	Continued From Previous Page																
348.4	SAND, some gravel, trace silt, occasional cobbles and boulders Compact to Dense Grey Wet		9	SS	5		350										
								349									
								348									
								347									
					11	SS	48		346								
									345								
									344								
					13	SS	42		343								
342.9	Boulder (300mm) from 15.3m to 15.6m		12	SS	100/ 0.075		345										
							344										
							343										
17.4	BEDROCK, porphyry, fresh, dark grey with quartz inclusions (light grey) Sub-vertical joint (25mm) at 17.1m, 17.5m Sub-vertical joint (25mm to 50mm) at 18.3m, 18.6m, 19.1m, 19.2m, 19.3m Horizontal joint at 19.5m Sub-vertical joint (25mm) at 19.4m 125mm at 19.7m Horizontal joint (25mm) at 19.8m		1	RUN			342										
				2	RUN			341									
				3	RUN												
340.4																	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity 20
15 10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-03

3 OF 3

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 904.5 E 382 908.8 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2014.06.04 - 2014.06.04 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
19.9	Continued From Previous Page END OF BOREHOLE AT 19.9m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND MIXED WITH CUTTINGS TO 0.3m, THEN CONCRETE TO 0.1m, THEN ASPHALT TO SURFACE.													

RECORD OF BOREHOLE No PRB-04

1 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 979.2 E 382 900.7 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.06.03 - 2014.06.03 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
360.5	GROUND SURFACE												
0.0	ASPHALT: (80mm)												
0.1	SAND and GRAVEL to Gravelly SAND, trace silt Loose to Compact Brown Moist to Wet (FILL)		1	GS									41 50 9 (SI+CL)
			1	SS	25								
			2	SS	7								
			3	SS	18								
			4	SS	14								
356.0													
4.5	PEAT, amorphous Firm Black Moist		5	SS	4								
354.9													
5.6	Silty CLAY Soft Grey Wet		6	SS	0								0 0 43 57
			7	SS	0								
351.2													
9.3	SILT, trace to some clay, trace sand Very Loose to Loose Grey Wet		8	SS	3								0 0 85 15

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-04

2 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 979.2 E 382 900.7 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.06.03 - 2014.06.03 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
<div><div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div><div><div>20406080100</div><div>W P W W L</div><div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div><div>WATER CONTENT (%)</div></div></div>														
	Continued From Previous Page													
	Sandy		9	SS	4		350							
							349							
348.6														
11.9	Silty SAND , trace gravel, occasional cobbles Compact to Very Dense Grey Moist		10	SS	13		348							
							347							
			11	SS	100/ 0.275									
							346							
			12	SS	50/ 0.100									
							345							
							344							
			13	SS	100/ 0.125									
							343							
342.0			14	SS	100/ 0.075		342							
18.5	END OF BOREHOLE AT 18.5m. BOREHOLE OPEN TO 16.0m AND WATER LEVEL AT 1.9m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG MIXED WITH CUTTINGS TO 0.2m, THEN CEMENT TO 0.1m, THEN ASPHALT TO SURFACE.													

2 68 30
(SI+CL)

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-05

1 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 982.1 E 382 910.8 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.05.31 - 2014.06.01 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
360.5	GROUND SURFACE													GR SA SI CL
0.0	SAND , trace to some gravel, trace silt Compact to Loose Brown Moist to Wet (FILL)						360							
			1	SS	23					○				
			2	SS	10		359			○				9 83 8 (SI+CL)
			3	SS	4		358			○				
			4	SS	3		357			○				10 87 3 (SI+CL)
357.0	PEAT , amorphous Black Moist						356					○		
355.8	Silty CLAY , trace shells and organics Soft Grey Wet		5	SS	1		355	3.0 +				○ 81		0 0 48 52
			6	SS	0		354					○		
			7	SS	2		353	8.0 +				○		
351.8	SILT , some sand to sandy, trace clay Very Loose to Loose Grey Wet		8	SS	1		352							
8.7							351			○				0 27 66 7

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-05

2 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 982.1 E 382 910.8 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.05.31 - 2014.06.01 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page							20	40	60	80	100				
			9	SS	7									○		
348.8																
11.7	Silty SAND , trace to some gravel, occasional cobbles Compact to Very Dense Grey Wet															
			10	SS	29									○		
			11	SS	100/ 0.150									○		
			12	SS	100/ 0.075									○		
343.3			13	SS	100/ 0.275									○		
17.2	END OF BOREHOLE AT 17.2m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jun 03/14 0.7 359.8 Jun 04/14 0.6 359.9															





ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 9/22/14

RECORD OF BOREHOLE No PRB-06

1 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 997.5 E 382 906.3 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.05.31 - 2014.05.31 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											
								20 40 60 80 100											
360.2	GROUND SURFACE																		
0.0	SAND , gravelly to trace gravel, trace silt Compact to Very Loose Brown Moist (FILL)		1	GS			360												
			2	SS	12			359											
			3	SS	17			358											
	4	SS	2		357														
	5	SS	3		356														
356.6																			
3.6	PEAT , amorphous Black Moist																		
355.9																			
4.3	Silty CLAY Soft to Firm Grey Moist Some silt seams		6	SS	1														
			7	SS	0														

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRB-06

2 OF 2

METRIC

WP# 6940-10-01 LOCATION Pelican River Bridge N 5 550 997.5 E 382 906.3 ORIGINATED BY SLL
 HWY 72 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.05.31 - 2014.05.31 CHECKED BY KS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	BOREHOLE OPEN TO 7.4m AND WATER LEVEL AT 1.8m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG MIXED WITH CUTTINGS TO SURFACE.													

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 9/22/14

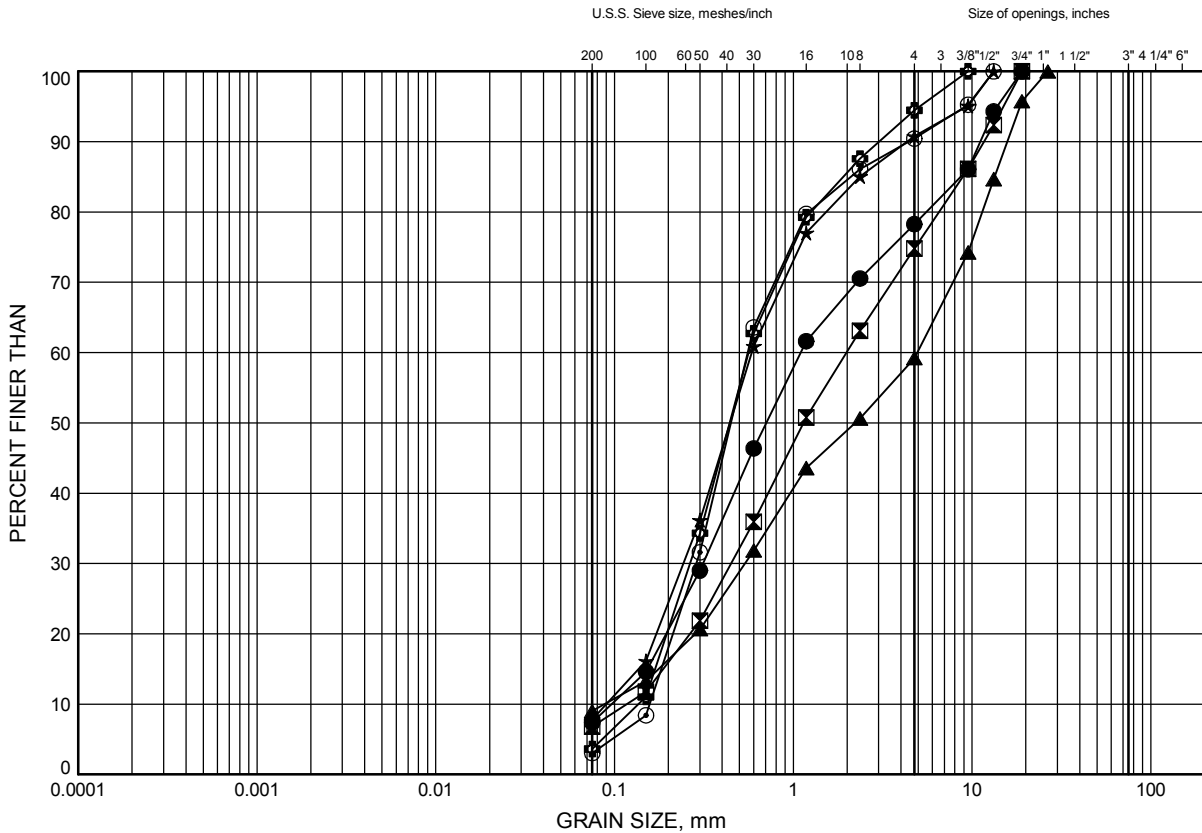
Appendix B

Laboratory Test Results

Pelican River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1a

SAND TO SAND & GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	0.15	360.05
⊠	PRB-03	1.07	359.23
▲	PRB-04	0.27	360.23
★	PRB-05	1.83	358.67
⊙	PRB-05	3.28	357.22
⊕	PRB-06	2.59	357.61

Date September 2014
WP# 6940-10-01

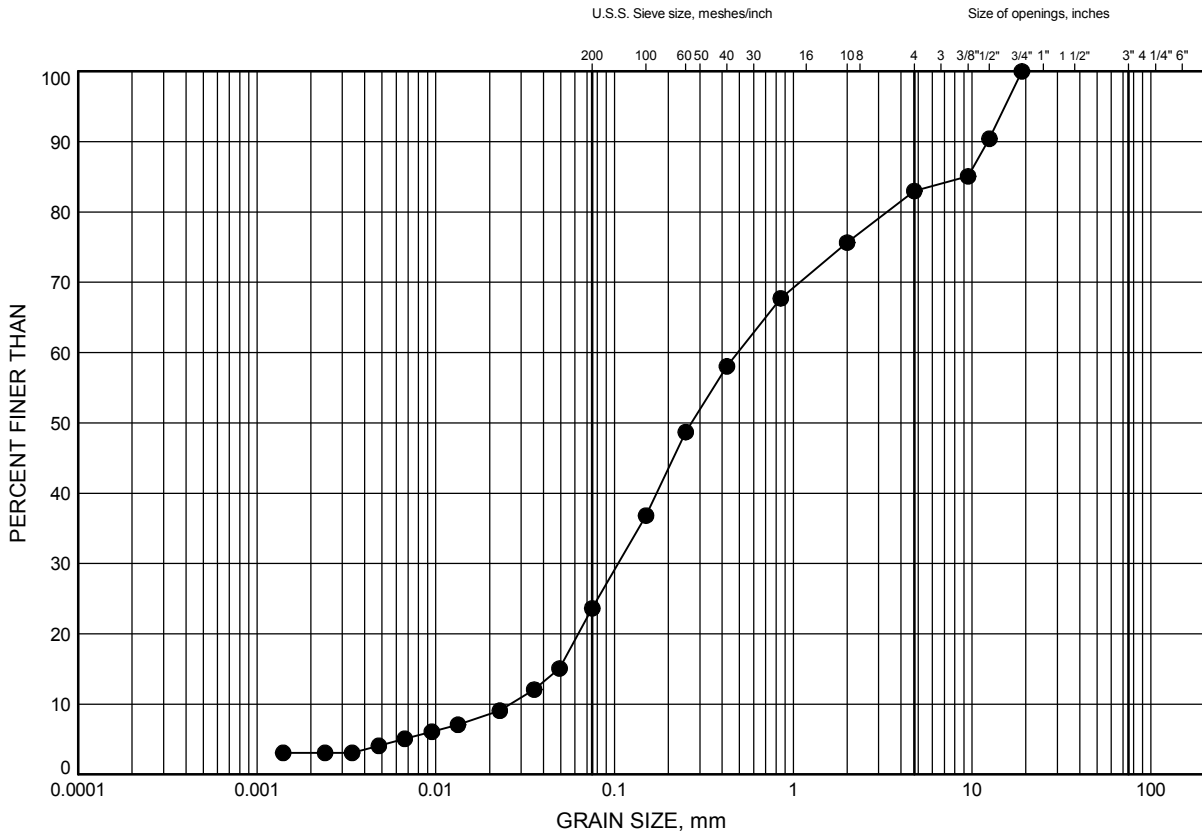


Prep'd AN
Chkd. KS

Pelican River Bridge
GRAIN SIZE DISTRIBUTION

FIGURE B1b

SILTY SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	2.54	357.66

Date September 2014
WP# 6940-10-01

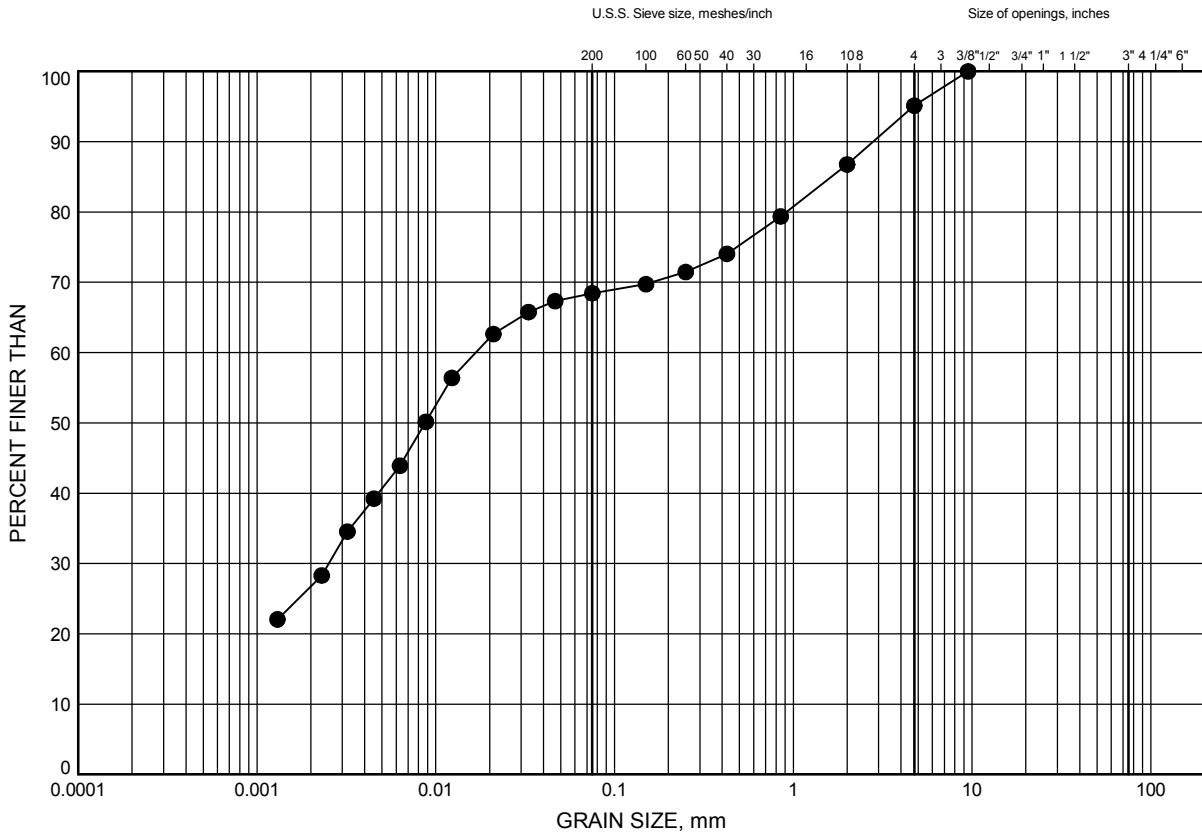


Prep'd AN
Chkd. KS

Pelican River Bridge
GRAIN SIZE DISTRIBUTION

FIGURE B1c

SANDY, CLAYEY SILT FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-02	1.83	358.47

Date September 2014
WP# 6940-10-01



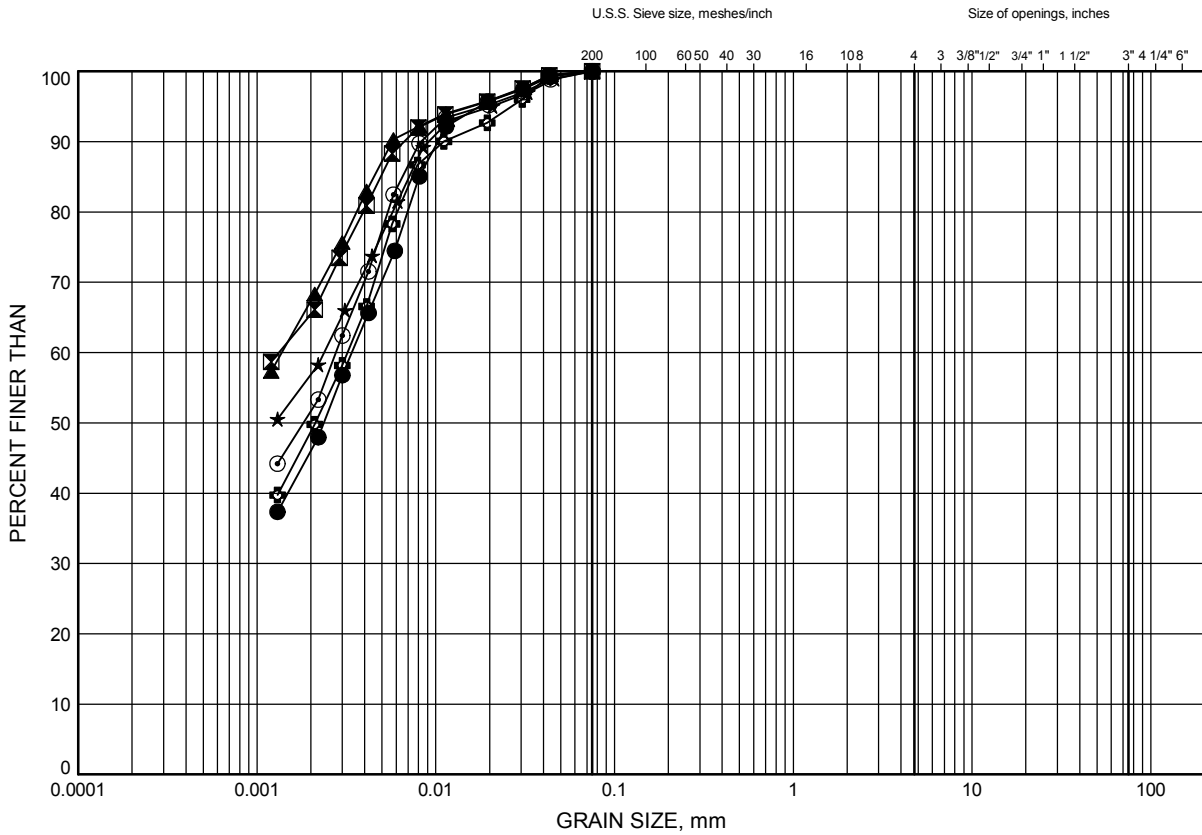
Prep'd AN
Chkd. KS

Pelican River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	4.88	355.32
⊠	PRB-02	6.40	353.90
▲	PRB-03	6.40	353.90
★	PRB-04	6.40	354.10
⊙	PRB-05	4.80	355.70
⊕	PRB-06	6.40	353.80

Date September 2014

WP# 6940-10-01



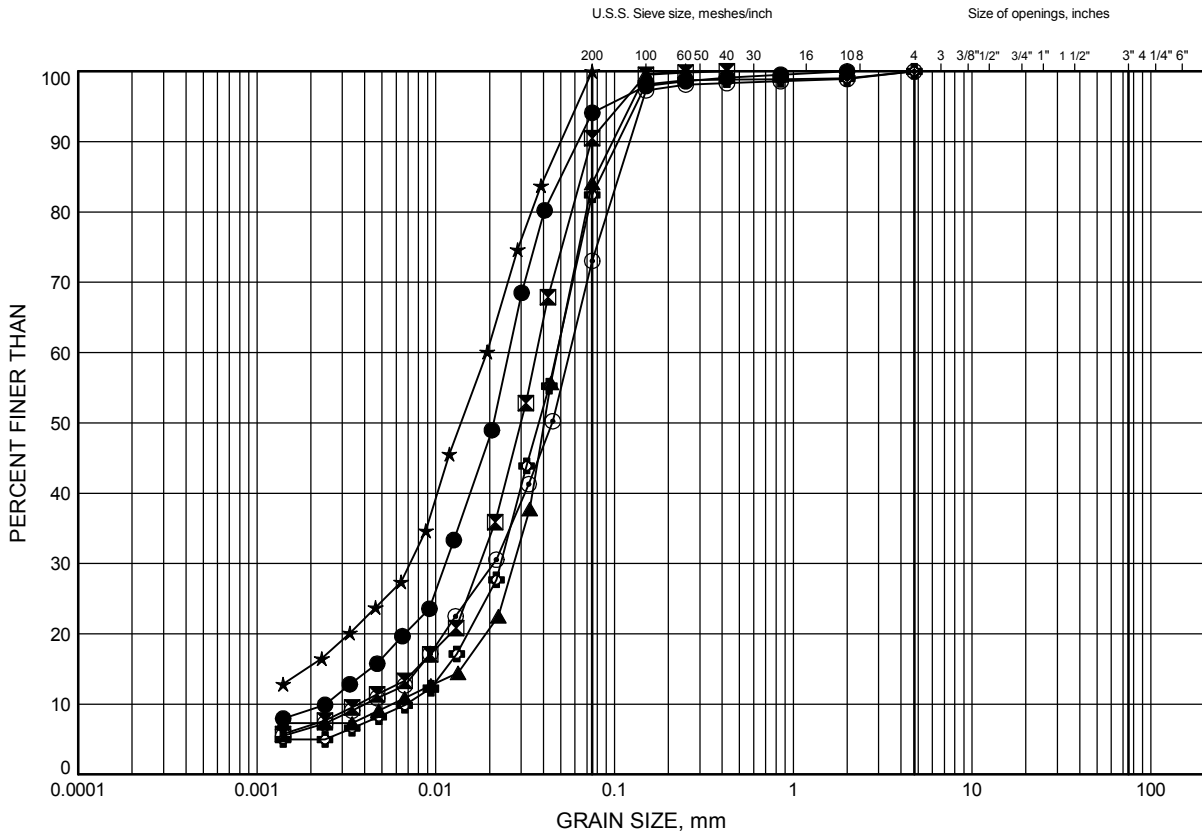
Prep'd AN

Chkd. KS

Pelican River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B3

SILT TO SANDY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	7.92	352.28
⊠	PRB-02	9.45	350.85
▲	PRB-03	9.45	350.85
★	PRB-04	9.53	350.97
⊙	PRB-05	9.45	351.05
⊕	PRB-06	9.45	350.75

Date September 2014

WP# 6940-10-01



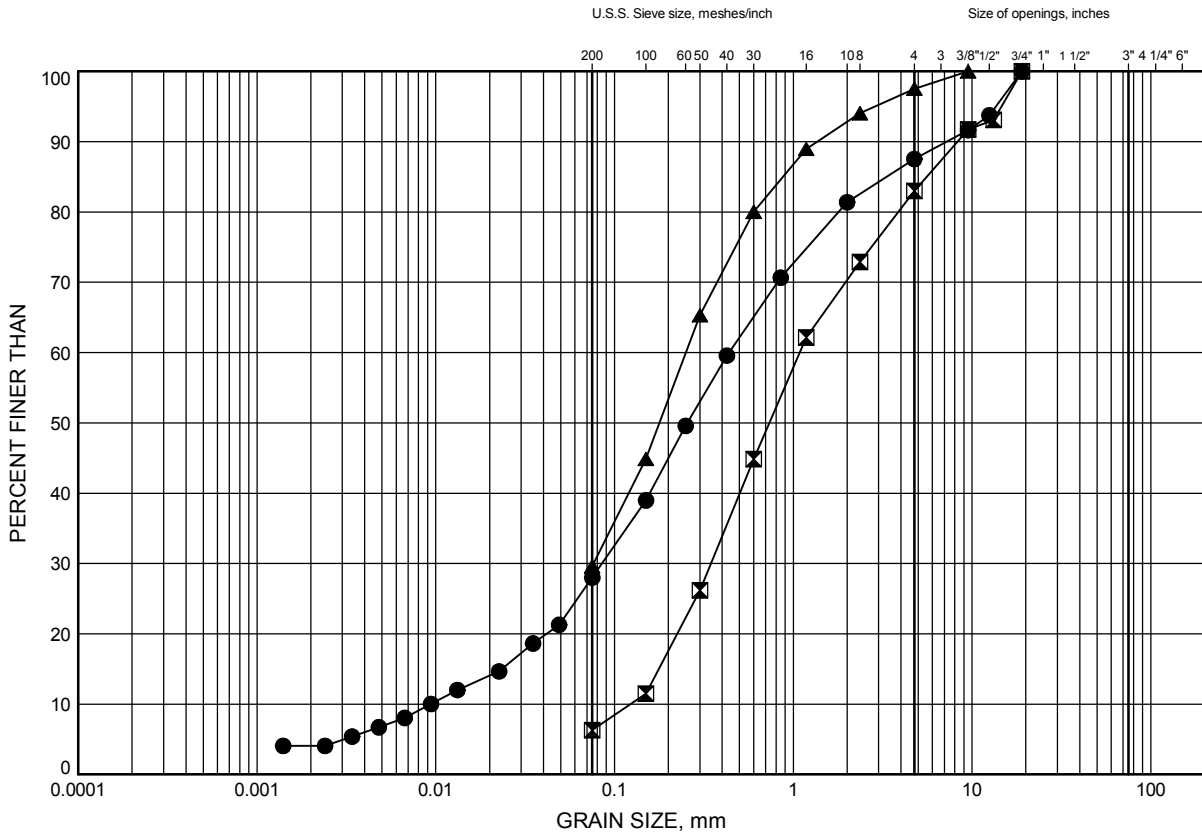
Prep'd AN

Chkd. KS

Pelican River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY SAND TO SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-02	12.50	347.80
⊠	PRB-03	12.50	347.80
▲	PRB-04	13.93	346.57

Date September 2014
WP# 6940-10-01

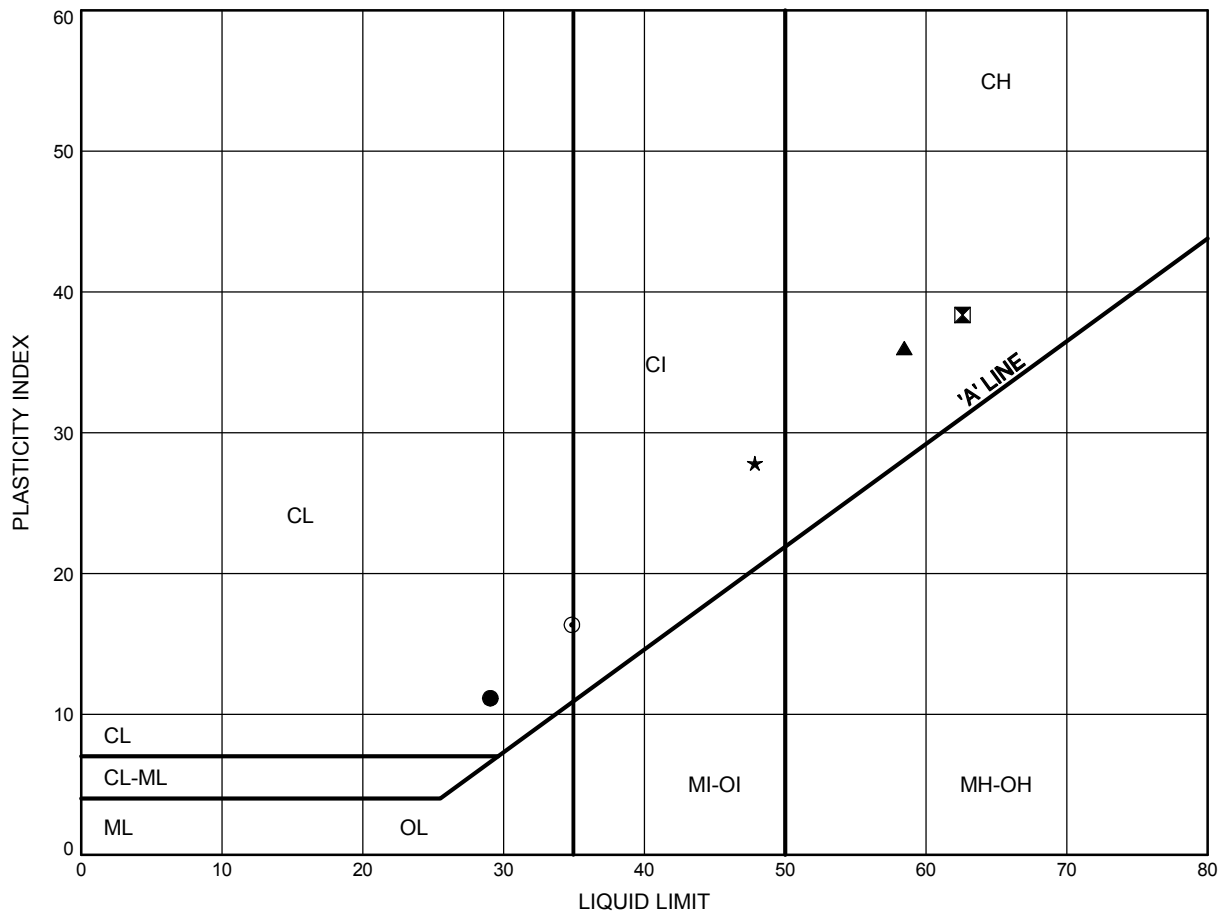


Prep'd AN
Chkd. KS

Pelican River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PRB-01	4.88	355.32
⊠	PRB-02	6.40	353.90
▲	PRB-03	6.40	353.90
★	PRB-04	6.40	354.10
⊙	PRB-06	6.40	353.80

Date September 2014
 WP# 6940-10-01



Prep'd AN
 Chkd. KS

Appendix C

Borehole Logs and Location Plan from Previous Investigation

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH	sampler advanced by static weight—weight, hammer
PH	sampler advanced by pressure—pressure, hydraulic
PM	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Relative Density	<i>N</i> , blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	<i>c_u</i> , lb./sq. ft.
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer ¹
Q	undrained triaxial ²
R	consolidated undrained triaxial ²
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_s	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume

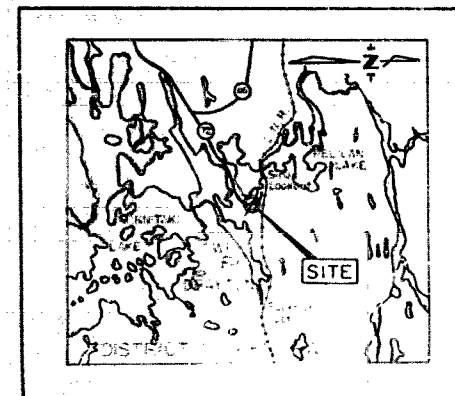
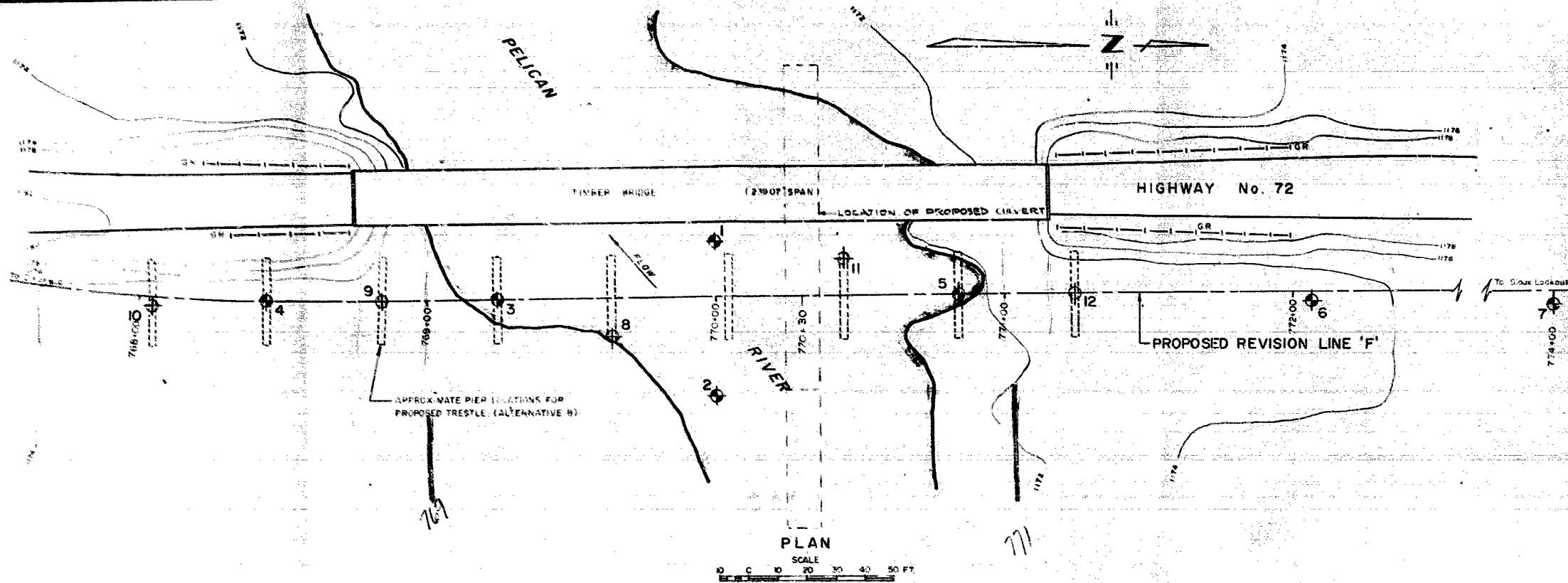
(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_e	coefficient of consolidation
T_v	time factor = $c_e t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_f	sensitivity

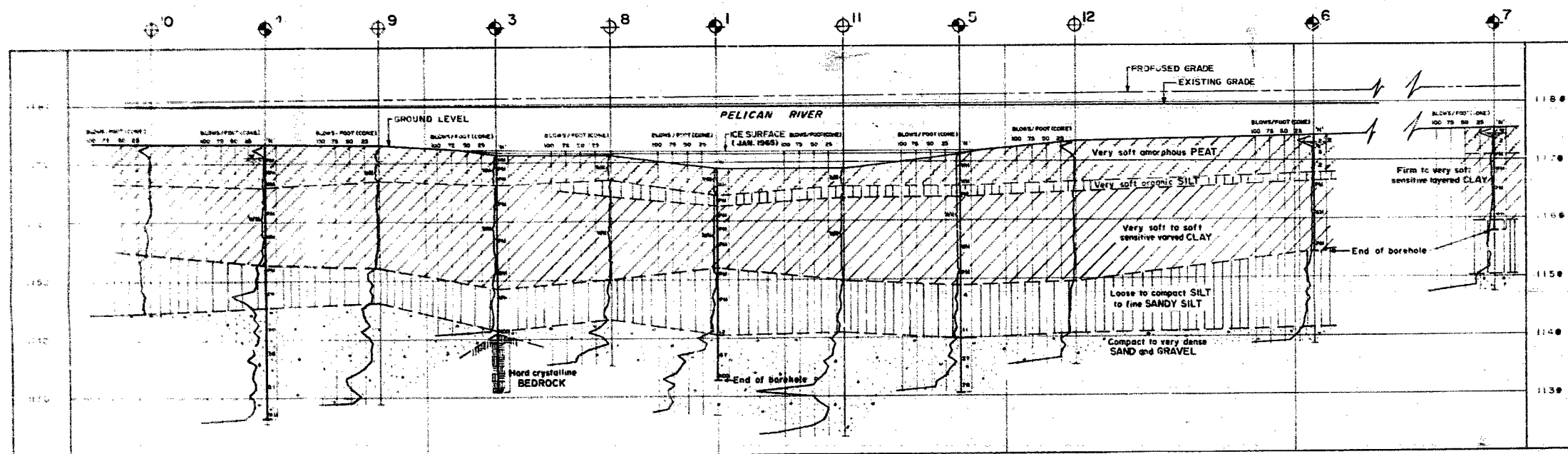
*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.



LEGEND

- Spot Elevation
- ⊕ Bore Hole
- ⊙ Bore Hole
- ⊖ Water at 10' depth

NO	ELEVATION	LOCATION
1	1171.9	345' LEFT
2	1171.9	770+00 345' RIGHT
3	1172.3	769+25 LINE 'F'
4	1173.2	768+44 LINE 'F'
5	1171.8	770+84 LINE 'F'
6	1174.3	772+07 3' RIGHT
7	1175.3	774+00 45' RIGHT
8	1172.0	769+63 135' RIGHT
9	1173.1	766+84 LINE 'F'
10	1173.4	766+65 3' RIGHT
11	1171.8	770+44 125' LEFT
12	1173.5	771+24 LINE 'F'



NOTE
No boundaries between soil strata have been established at Bore Hole locations. Between Bore Holes the boundaries are obtained from geological evidence and may be subject to modification.

H.Q. GOLDER & ASSOCIATES LIMITED
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

PELICAN RIVER
KING'S HIGHWAY NO. 72
DISTRICT OF KENORA
TWP. OF DRAYTON LOT --- CON ---

BORING PLAN AND SOIL STRATIGRAPHY SECTION

CHECKED	APR 15 1965	41-9
CHECKED	APR 15 1965	41-9

DATE APRIL 15, 1965 SITE NO. 41-9

05753-2

SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS

RECORD OF BOREHOLE 2

LOCATION

See Figure 1

BORING DATE JAN. 4 & 5, 1965

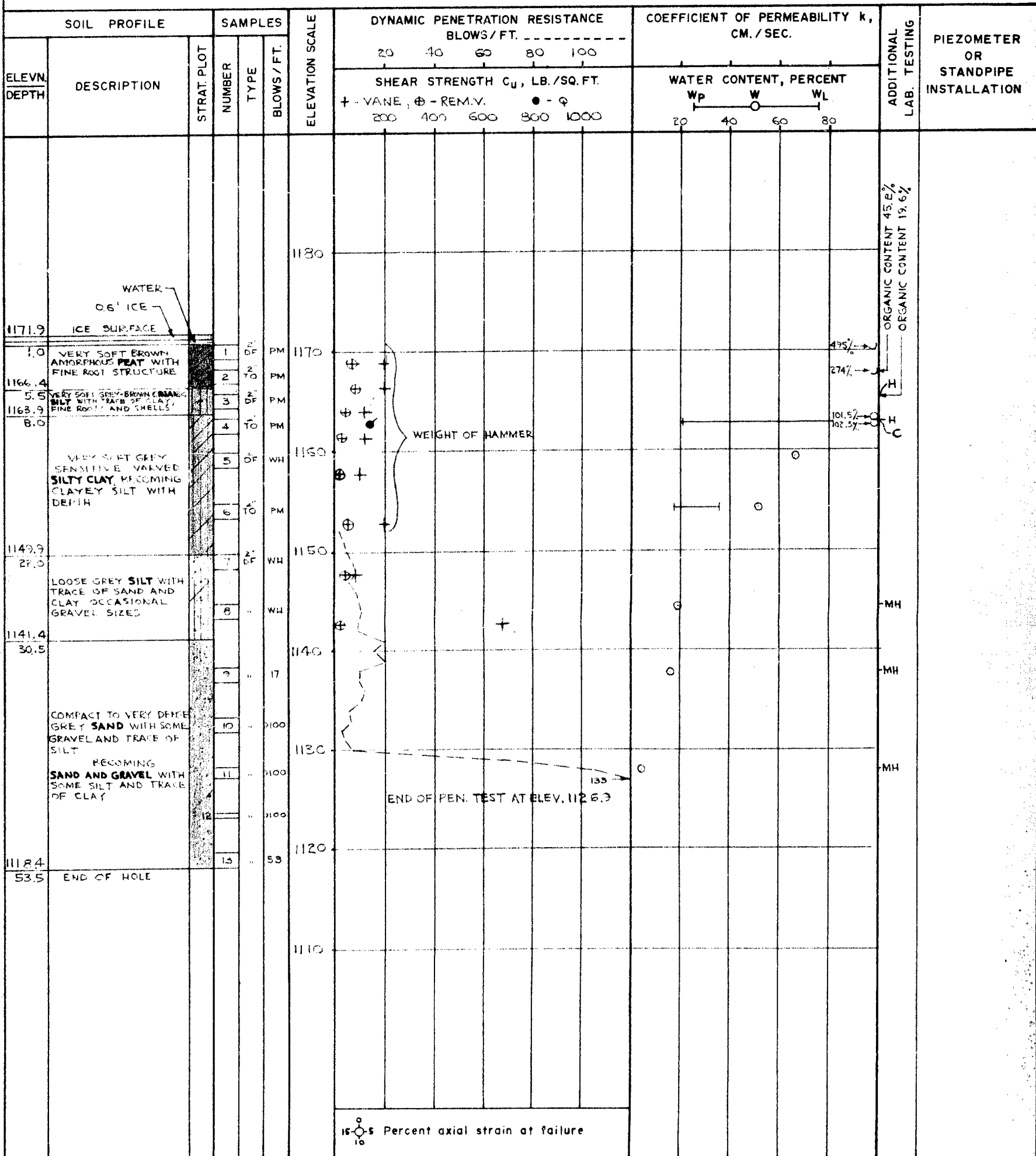
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



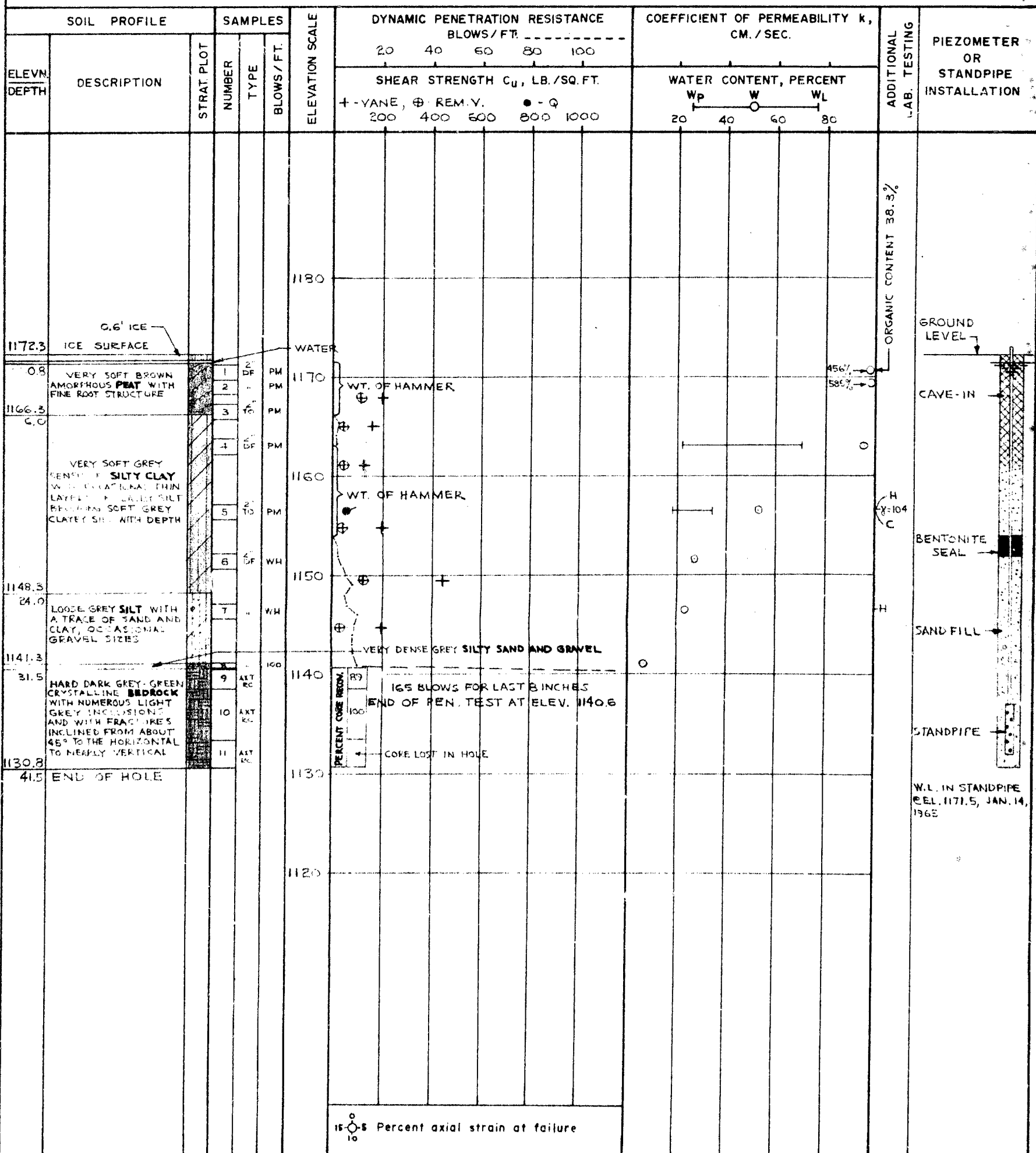
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN B.H. & M.J.
CHECKED

RECORD OF BOREHOLE 3

LOCATION See Figure 1 BORING DATE JAN. 6-7, 1965 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX 8 AX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10' 0"

GOLDER & ASSOCIATES

DRAWN R.H.S. 1/10/65
CHECKED

RECORD OF BOREHOLE 4

LOCATION See Figure 1

BORING DATE JAN. 8-10, 1965

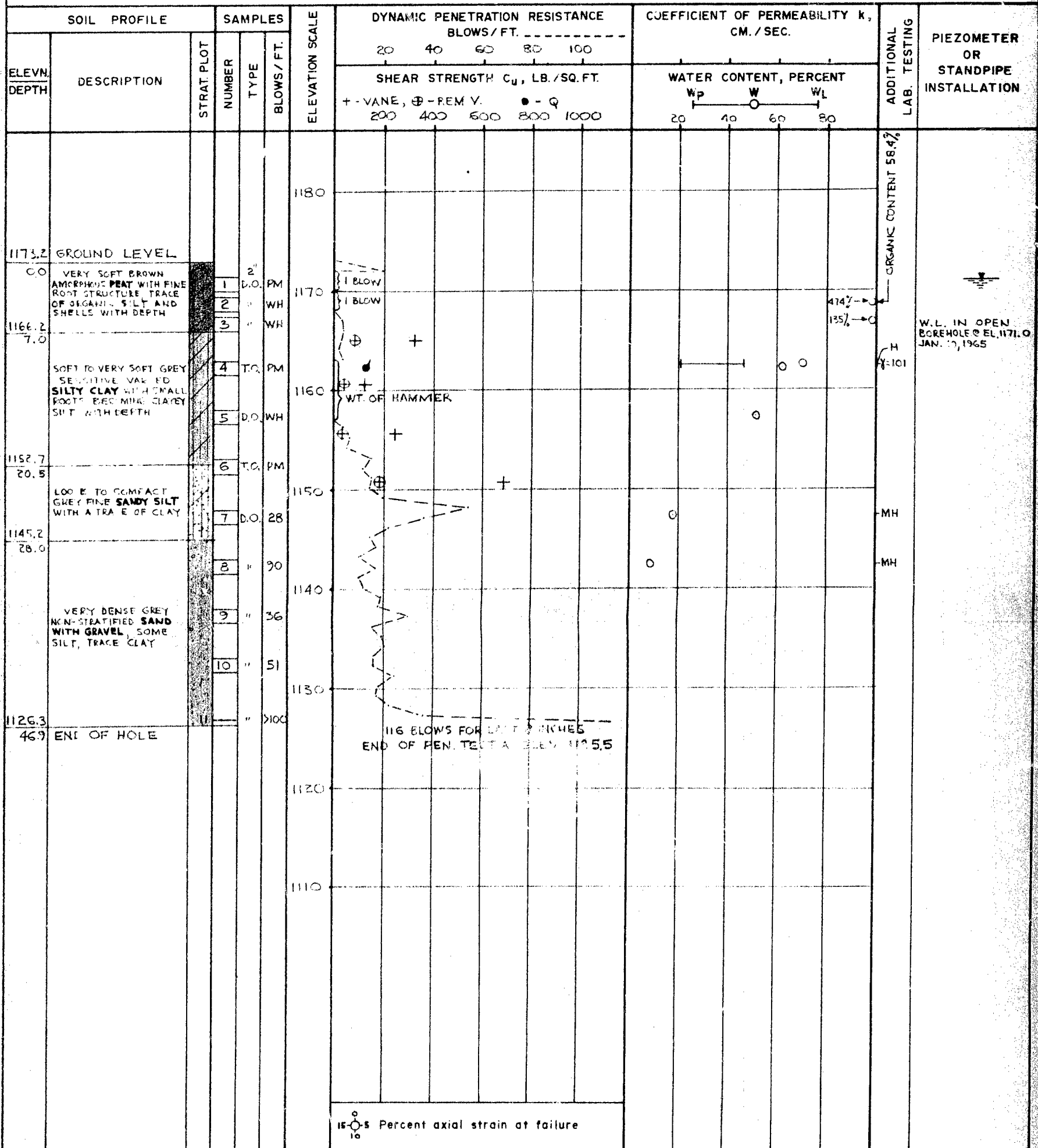
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *Ward*
CHECKED *Ward*

RECORD OF BOREHOLE 5

LOCATION See Figure 1

BORING DATE JAN. 11-12, 1965

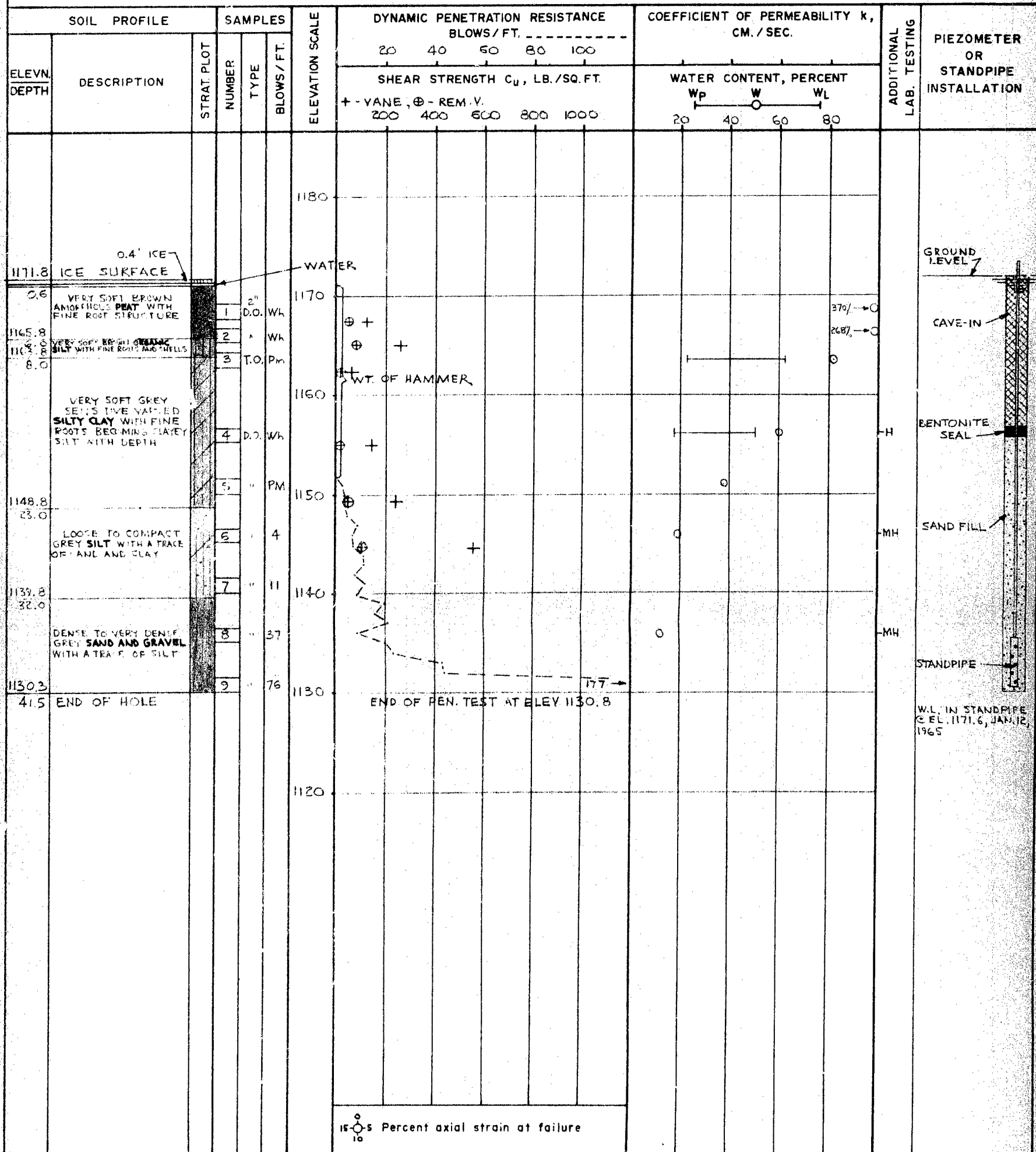
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *th (1)*
CHECKED *th*

RECORD OF BOREHOLE 9

LOCATION

See Figure 1

BORING DATE JAN. 7, 1965

DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ————	COEFFICIENT OF PERMEABILITY k , CM./SEC.		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20 40 60 80 100	WATER CONTENT, PERCENT W_p — W — W_L			
1173.1	GROUND LEVEL										
0.0	PROBABLY VERY SOFT AMORPHOUS PEAT										
1167.1											
6.0											
	PROBABLY VERY SOFT LAYERED CLAY										
1152.1											
21.0											
	PROBABLY LOOSE TO COMPACT FINE SANDY SILT										
1146.1											
27.0											
	PROBABLY COMPACT TO VERY DENSE SAND AND GRAVEL										
1128.7											
44.4	END OF PEN. TEST										

15-0-5 Percent axial strain at failure

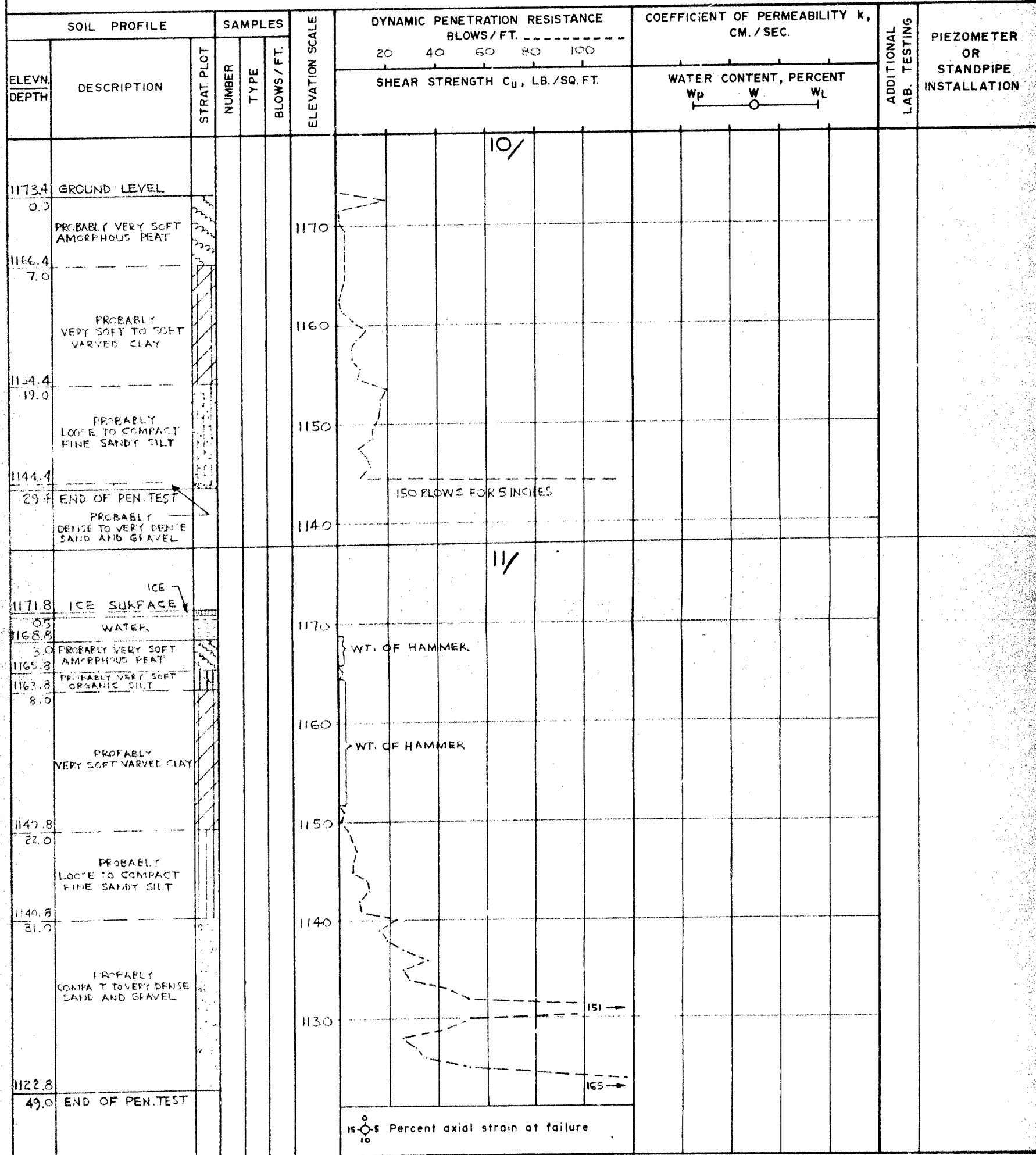
VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *msw*
CHECKED *msw*

RECORD OF BOREHOLES 10 & 11

LOCATION See Figure 1 BORING DATE JAN. 11, 1965 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



RECORD OF BOREHOLE 12

See Figure 1

BORING DATE JAN. 12, 1965

DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

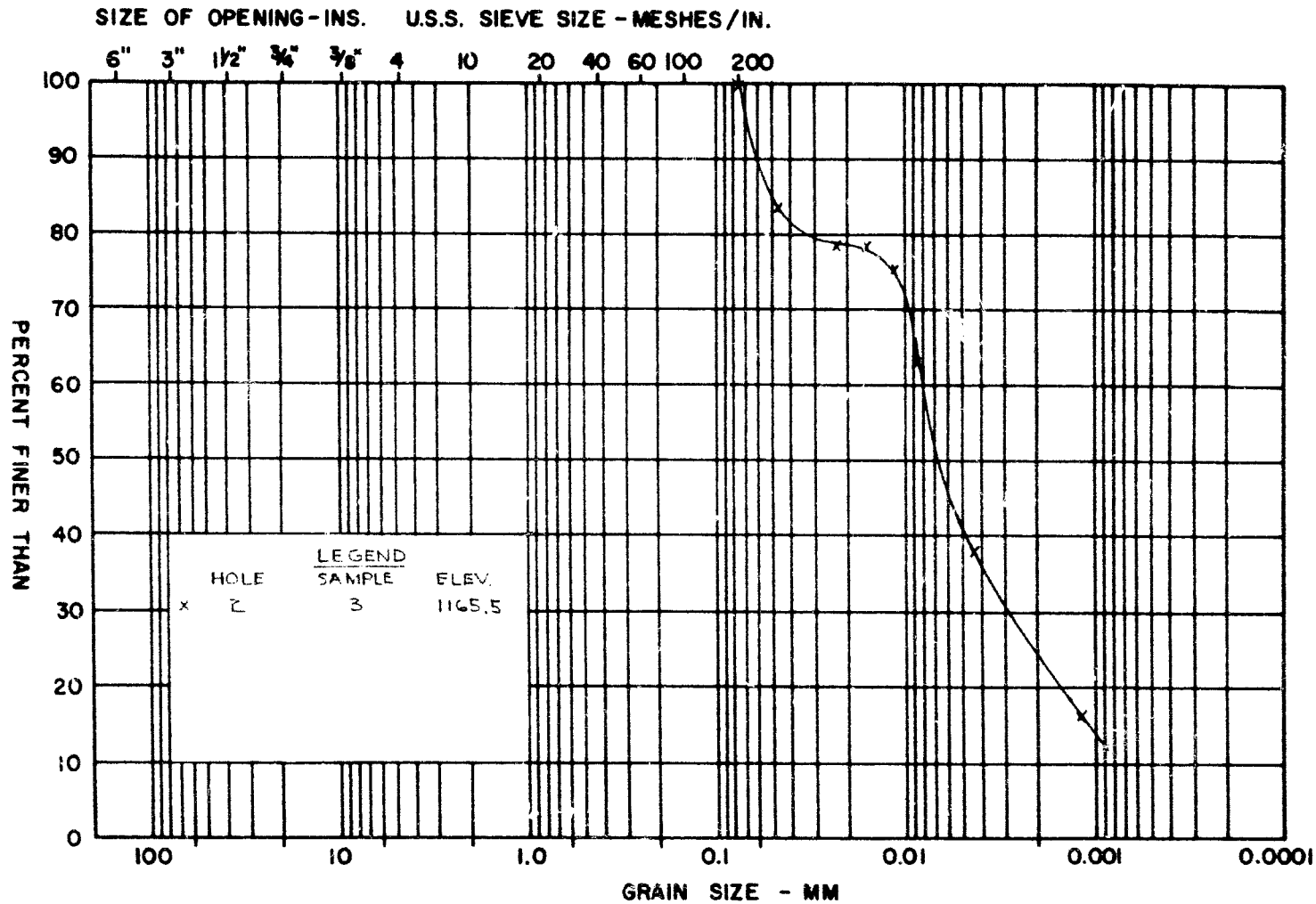
[illegible]

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN lms
CHECKED ll

M.I.T. GRAIN SIZE SCALE

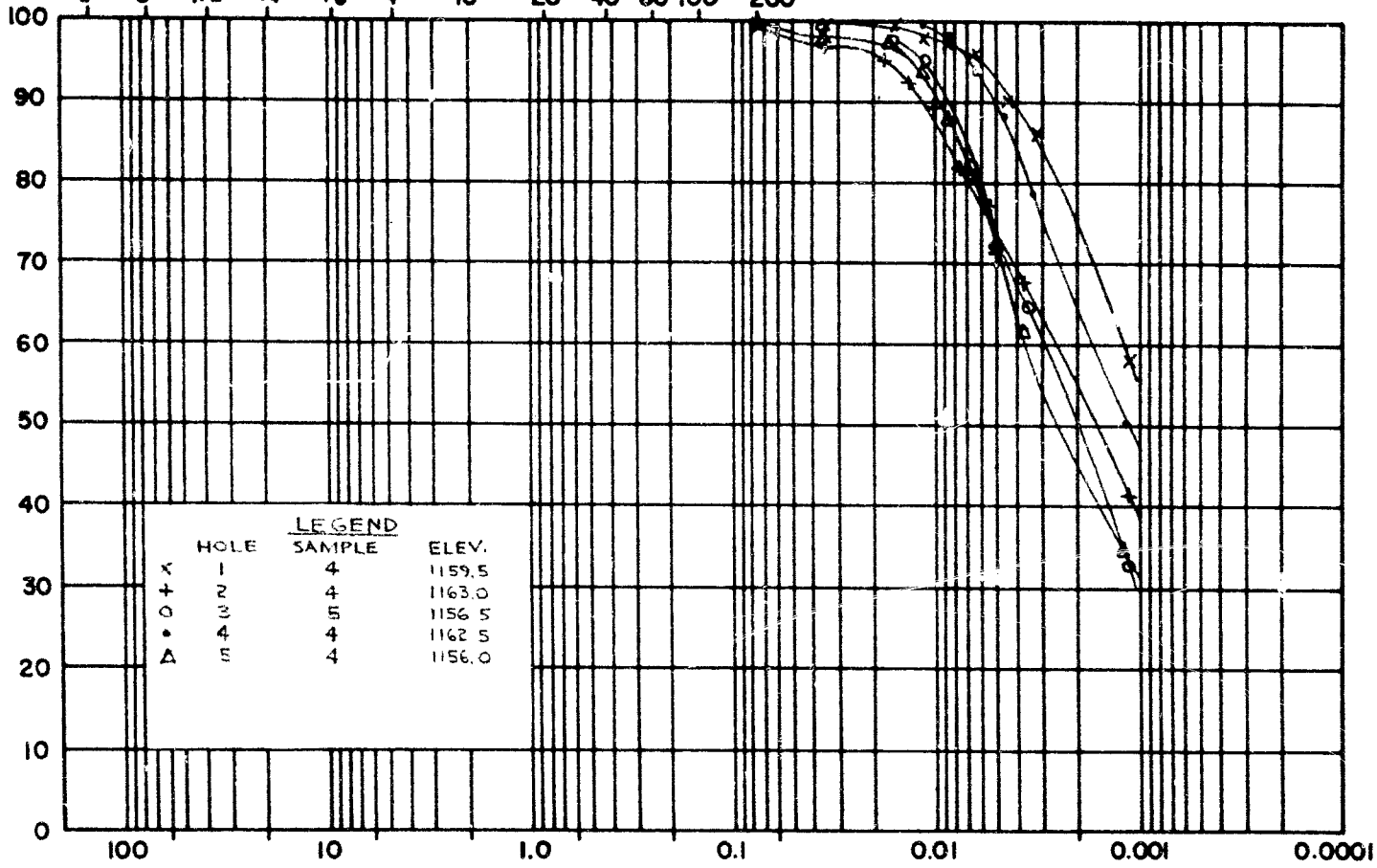


M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S. SIEVE SIZE - MESHES/IN.

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200

PERCENT FINER THAN



GRAIN SIZE - MM

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

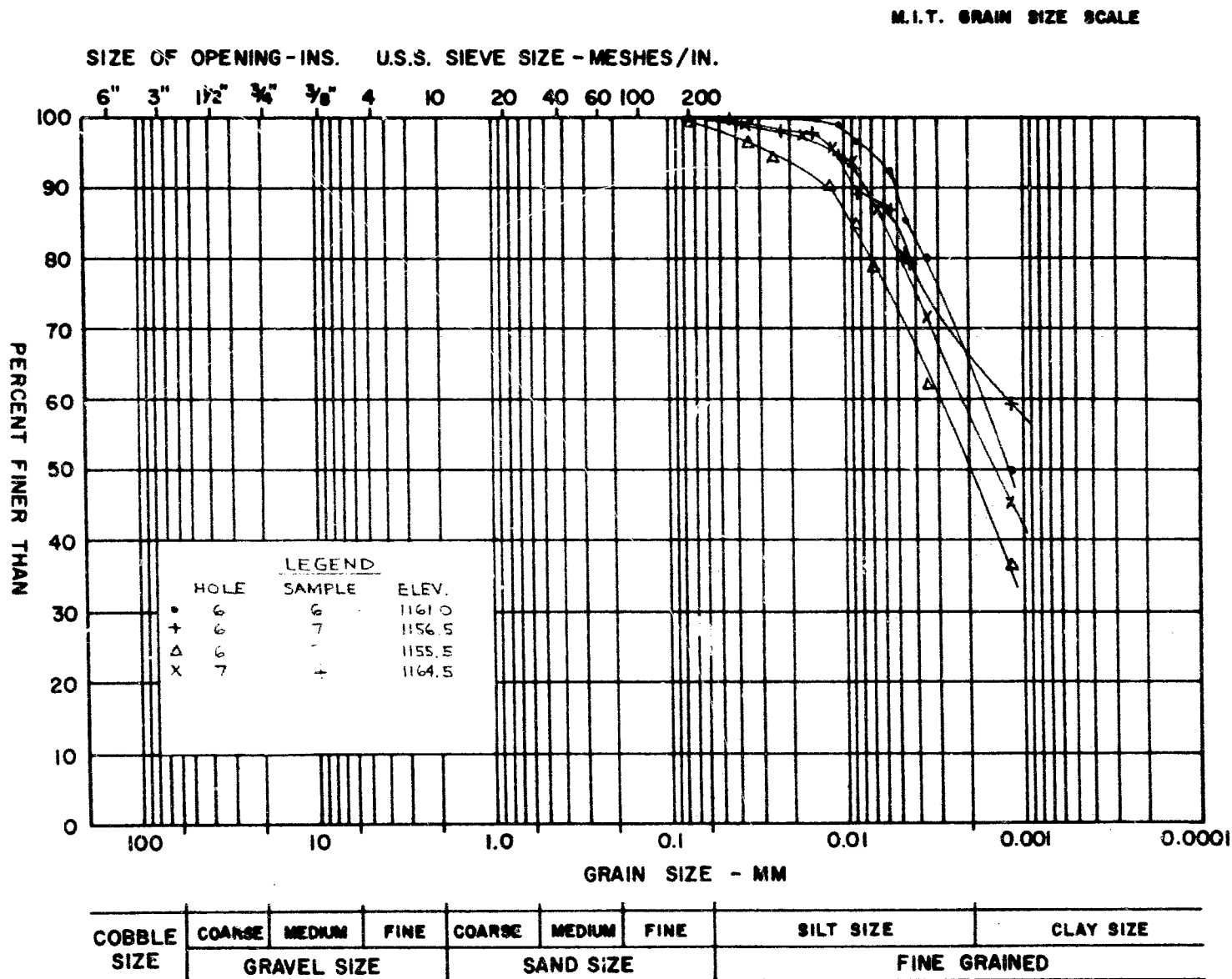
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SILTY CLAY

FIGURE 3

GRAIN SIZE DISTRIBUTION SILTY CLAY

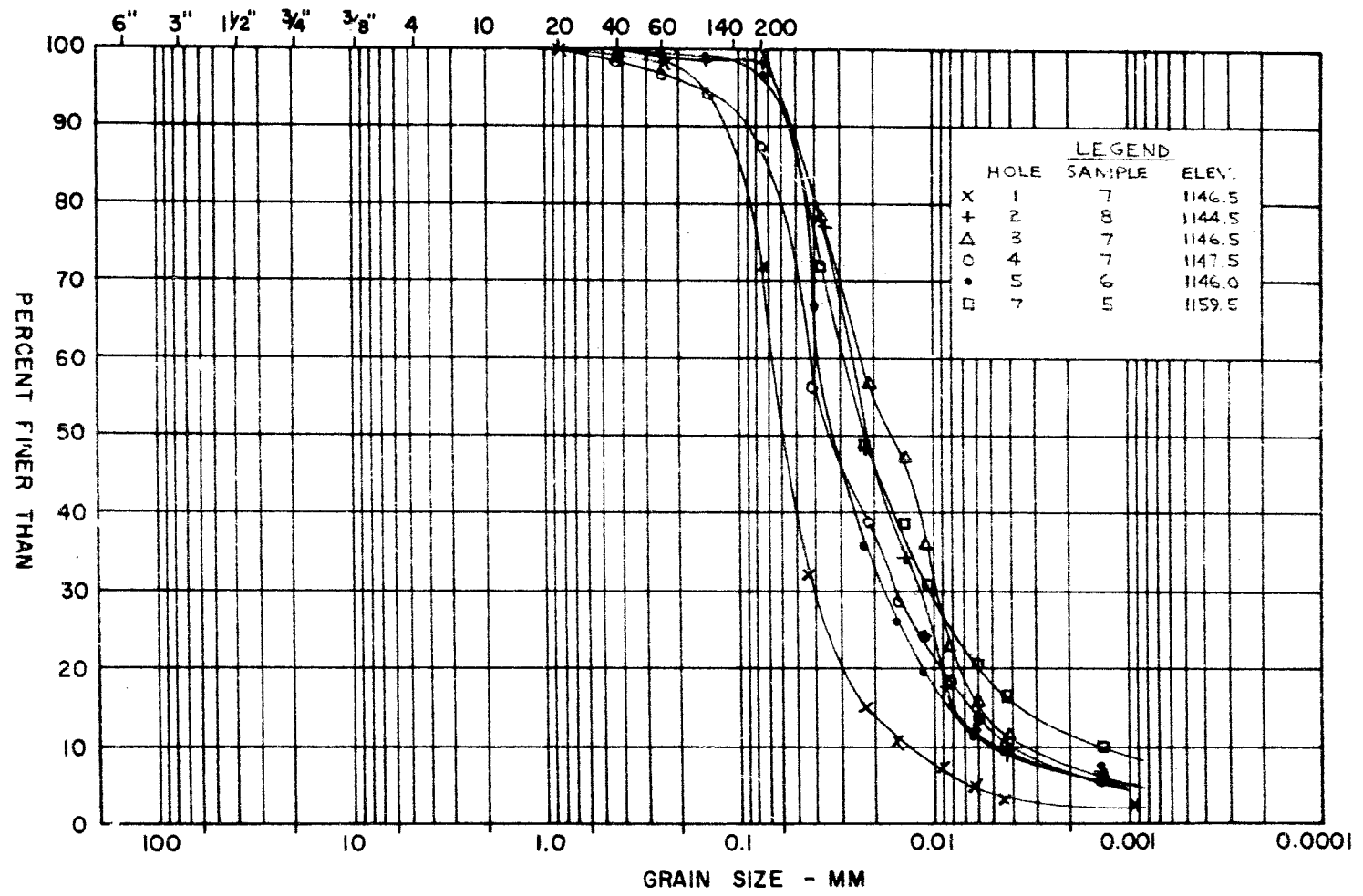
FIGURE 4



GOLDER & ASSOCIATES

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.



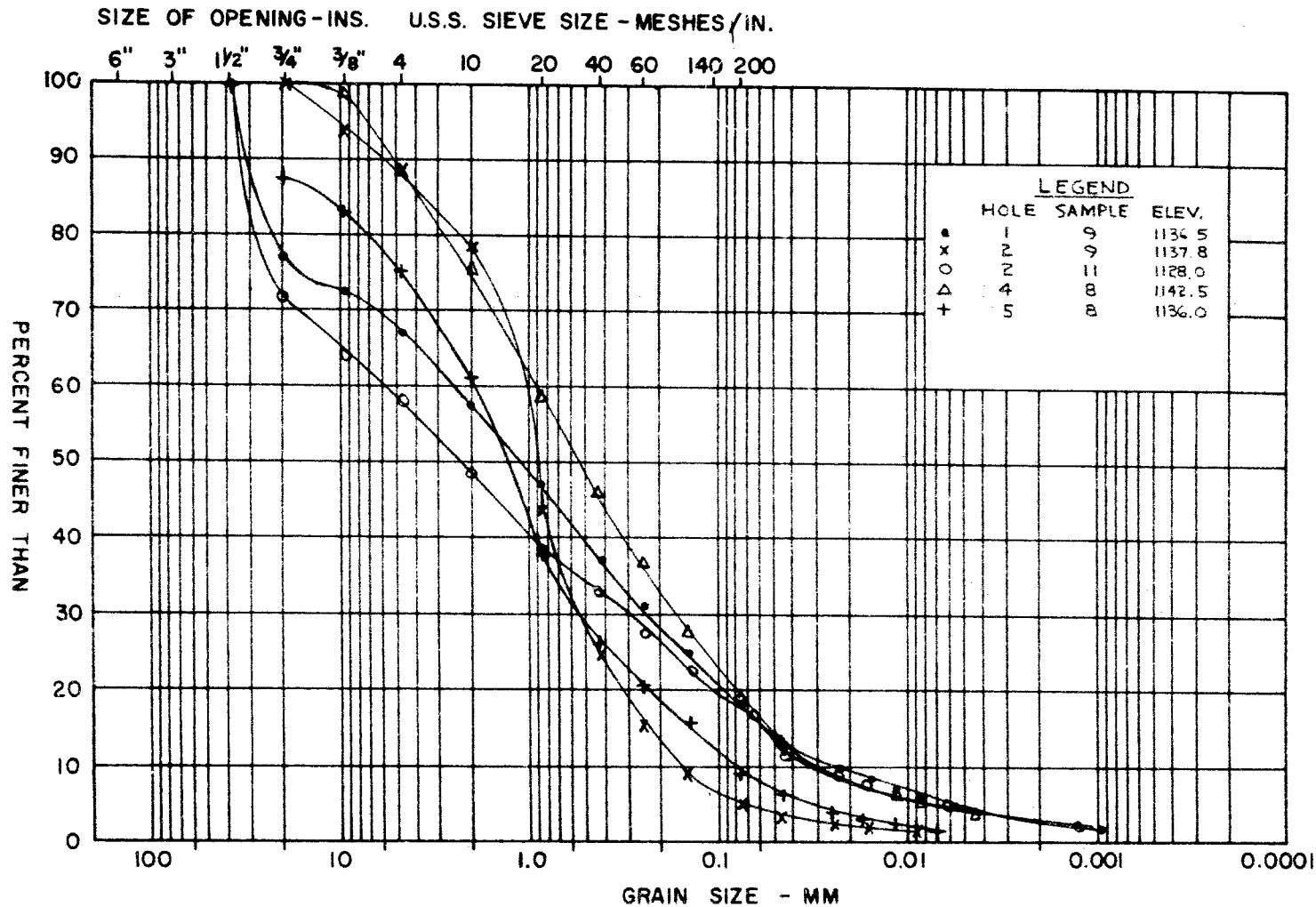
GOLDER & ASSOCIATES

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION
SILT TO SANDY SILT

FIGURE 5

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SAND AND GRAVEL

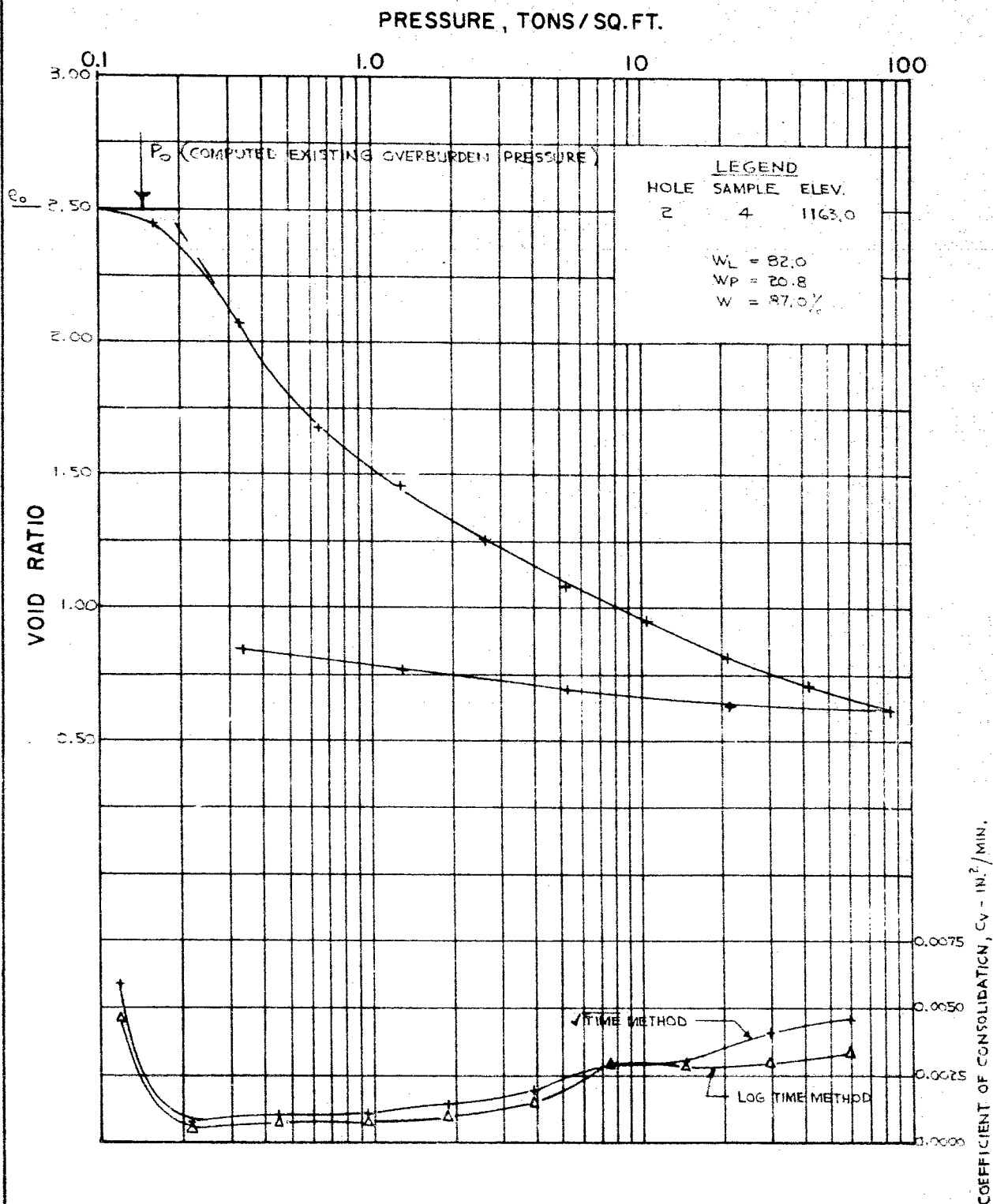
FIGURE 6

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

PROJECT No. 64156

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

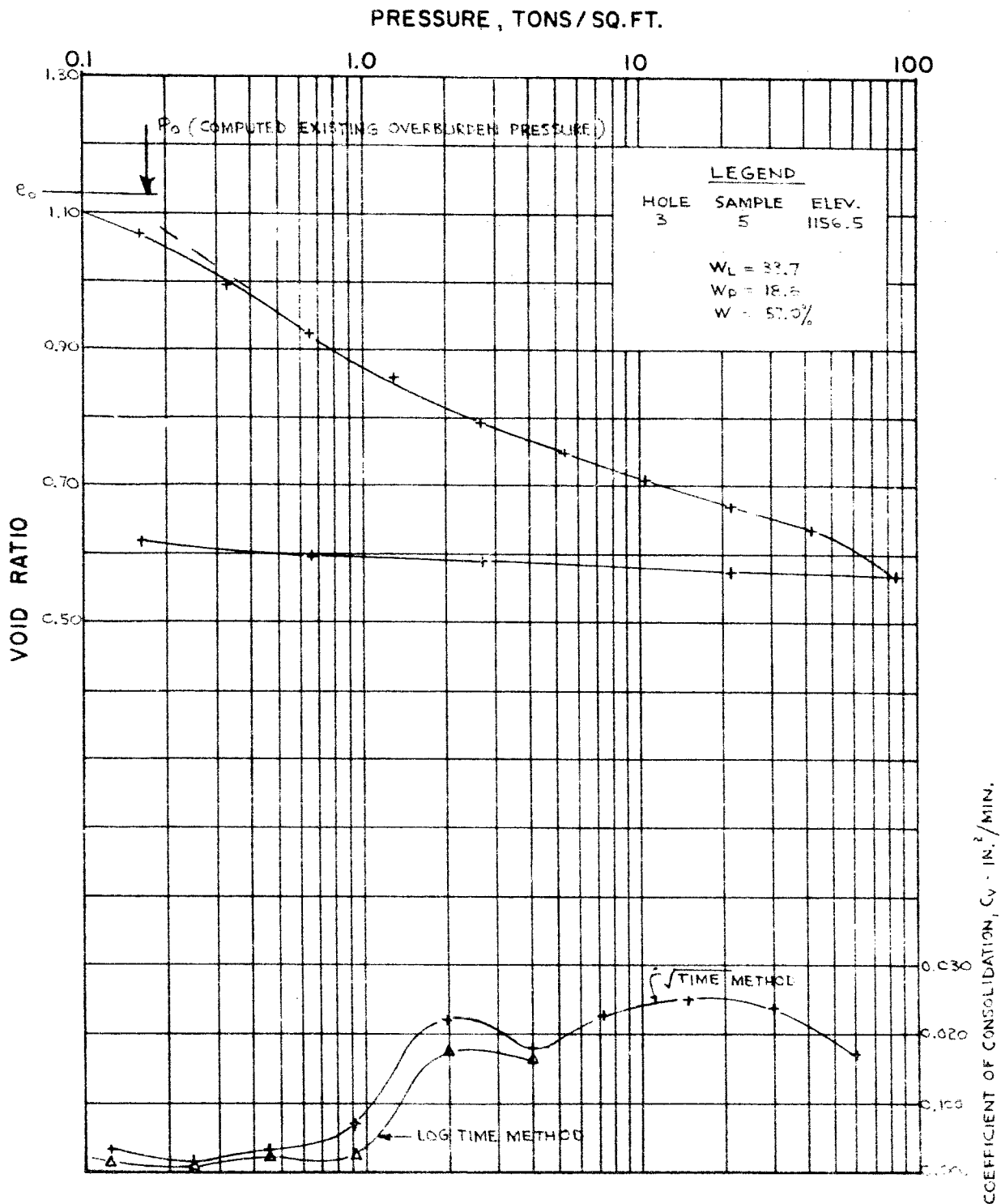
FIGURE 7



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

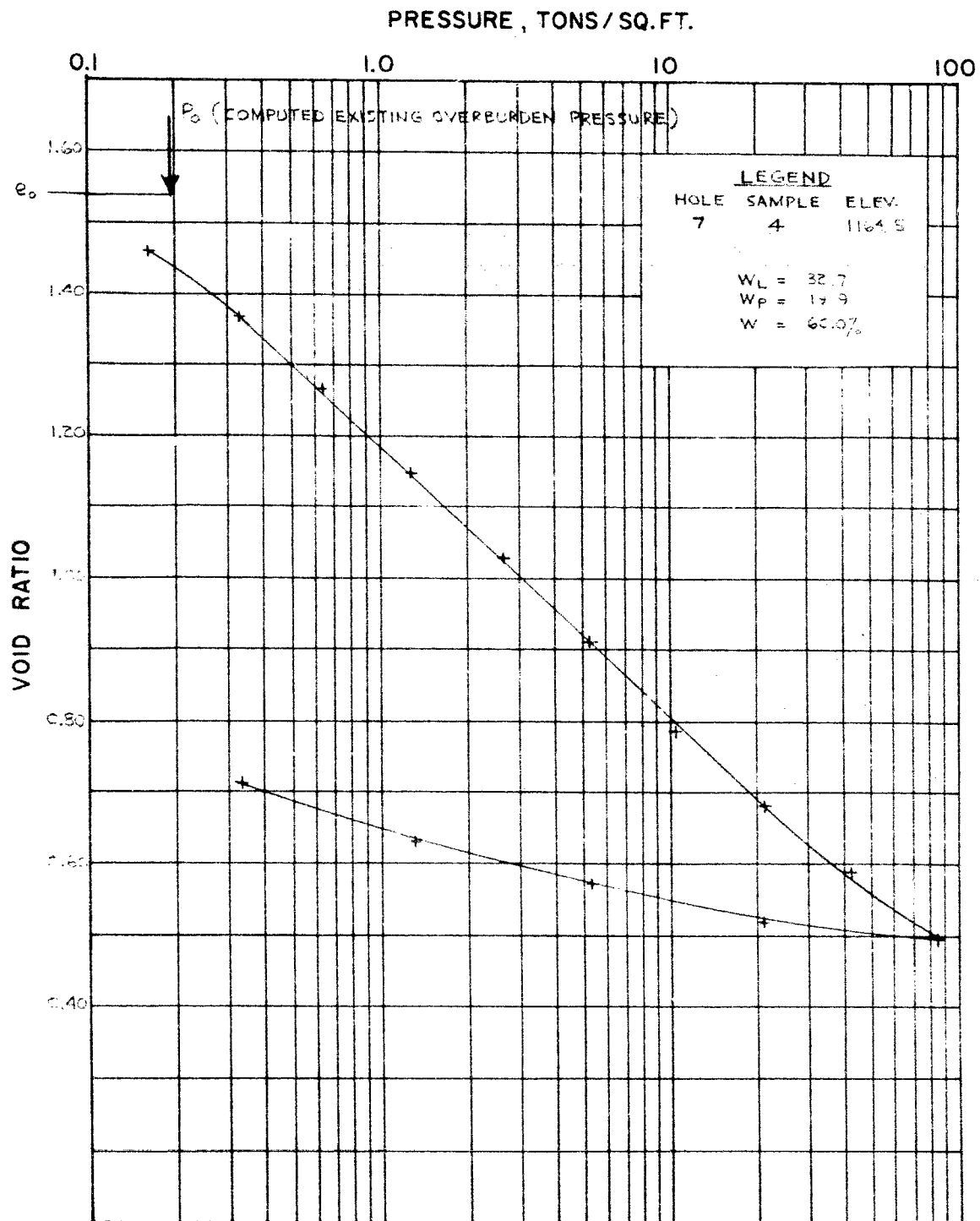
FIGURE 8



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 9



GOLDER & ASSOCIATES

Appendix D

Site Photographs



Looking South from North Abutment (High River Level - May 27, 2014)



Looking North towards Town of Sioux Lookout



Looking Southeast from North Abutment (Normal River Level - May 6, 2014)



Looking South from North Abutment (High River Level - May 27, 2014)

Appendix E
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil	Footings on Engineered Fill	Driven Piles	Caissons
<p>Advantages</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Not feasible for piers in river. ii. Low geotechnical resistance is available in native soils at abutments. iii. Potential for consolidation settlement in peat and silty clay. iv. Dewatering may be required, depending on depth of excavation. <p style="text-align: center;">NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. Allows use of perched abutments. iii. Higher geotechnical resistance than on native soil. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Cost of engineered fill placement. ii. Not feasible for piers in river. iii. Potential for consolidation settlement in peat and silty clay. iv. Dewatering may be required, depending on depth of excavation. <p style="text-align: center;">NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance on bedrock or very dense soils. ii. Piles can be driven into riverbed from the water surface. iii. Installation of piles could continue in freezing weather. iv. Allows integral abutment design. v. Requires less excavation than footings. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Possibility that cobbles and/or boulders may be encountered in the fill and native deposits. <p style="text-align: center;">RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded on bedrock. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Temporary liners will be required to install caissons in cohesionless soils below the river and groundwater levels. ii. Difficulty in sealing liners at bedrock surface. iii. Bedrock surface was not established at the pier and north abutment. iv. Possibility of cobbles and boulders being encountered during augering and liner installation. v. Difficulty in cleaning and inspecting bases. <p style="text-align: center;">NOT RECOMMENDED</p>

Appendix F

List of Standard Specifications and Special Provisions

1) The following Standard Specifications and Special Provisions are referenced in this report:

OPSS 501

OPSS 539

OPSS 804

OPSS 902

OPSS 903

OPSS.PROV 1010

OPSS.PROV 206

2) Recommended wording for “NSSP – Use of Heavy Construction Equipment”

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing and erection of the new bridge. The impact of the heavy equipment loads on the underlying sensitive soils, river banks and existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO’s RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

The assessment shall include, but not be limited to, the following:

- Determining appropriate setbacks for heavy equipment from the river banks and existing foundations;
- Evaluating the need for preventing heavy equipment from travelling or operating on the areas adjacent to the river, possibly requiring restriction of heavy loads to the existing highway embankment platform;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

Appendix G
Slope Stability Analysis

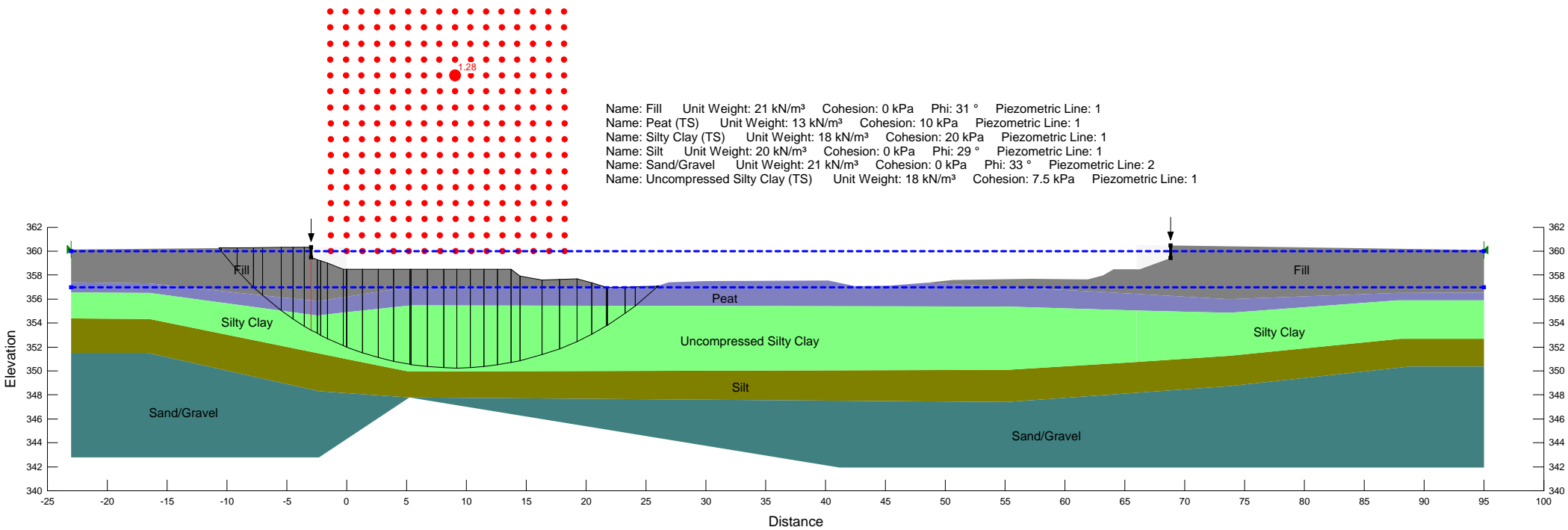


FIGURE 1

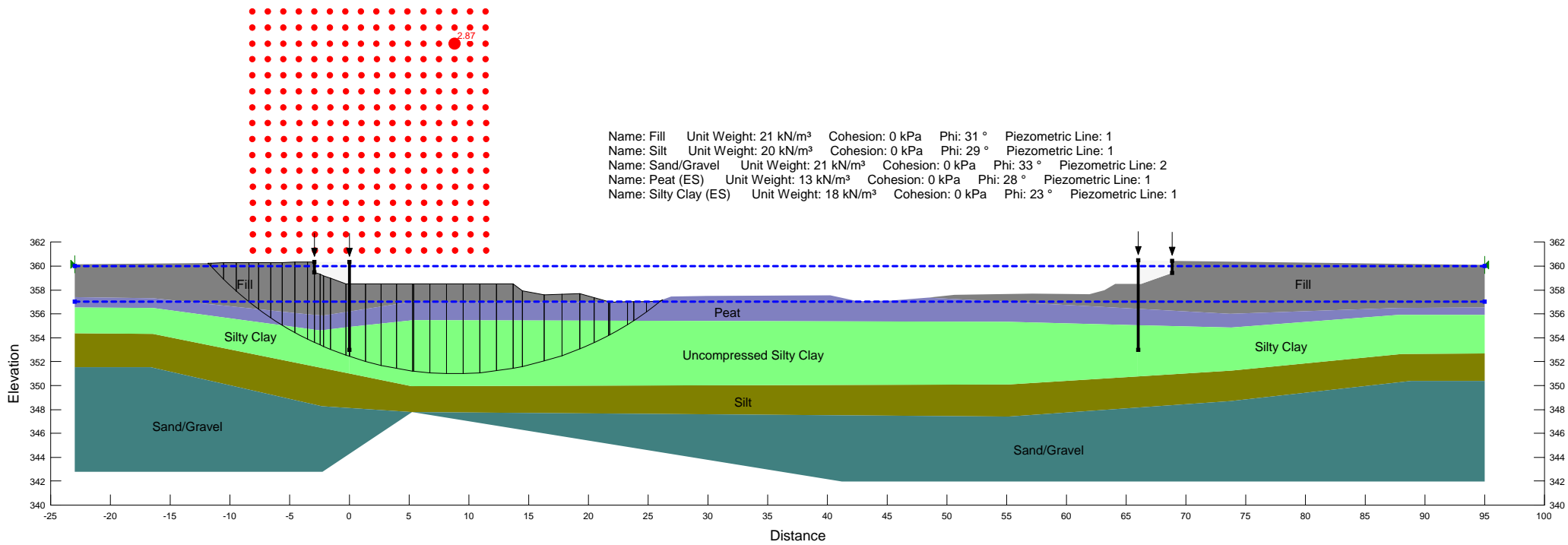


FIGURE 2

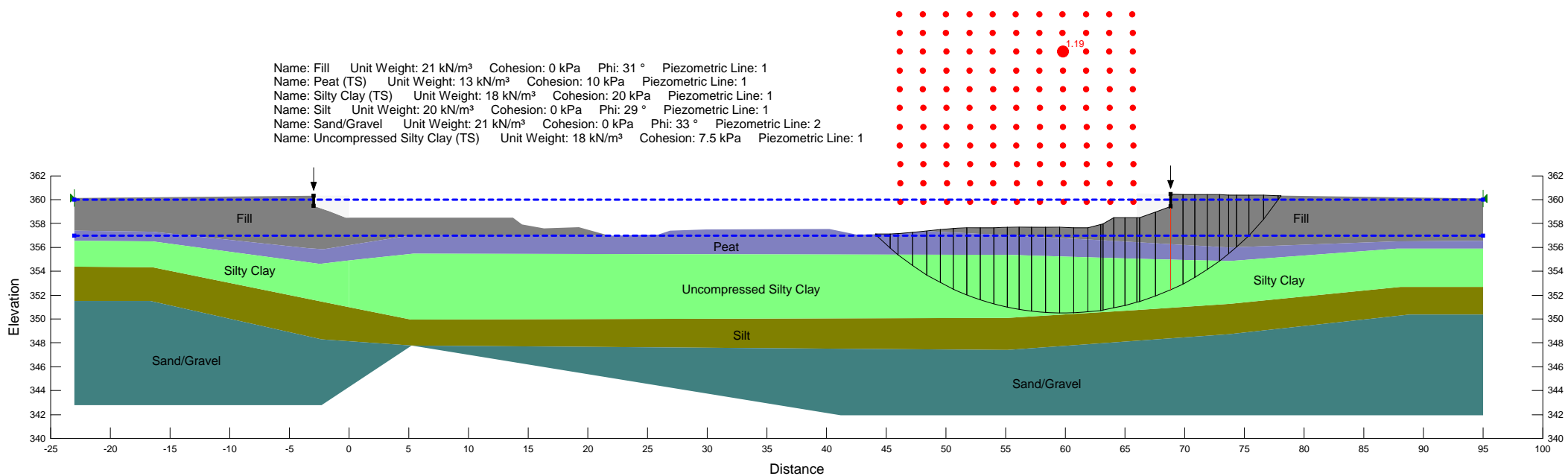


FIGURE 3

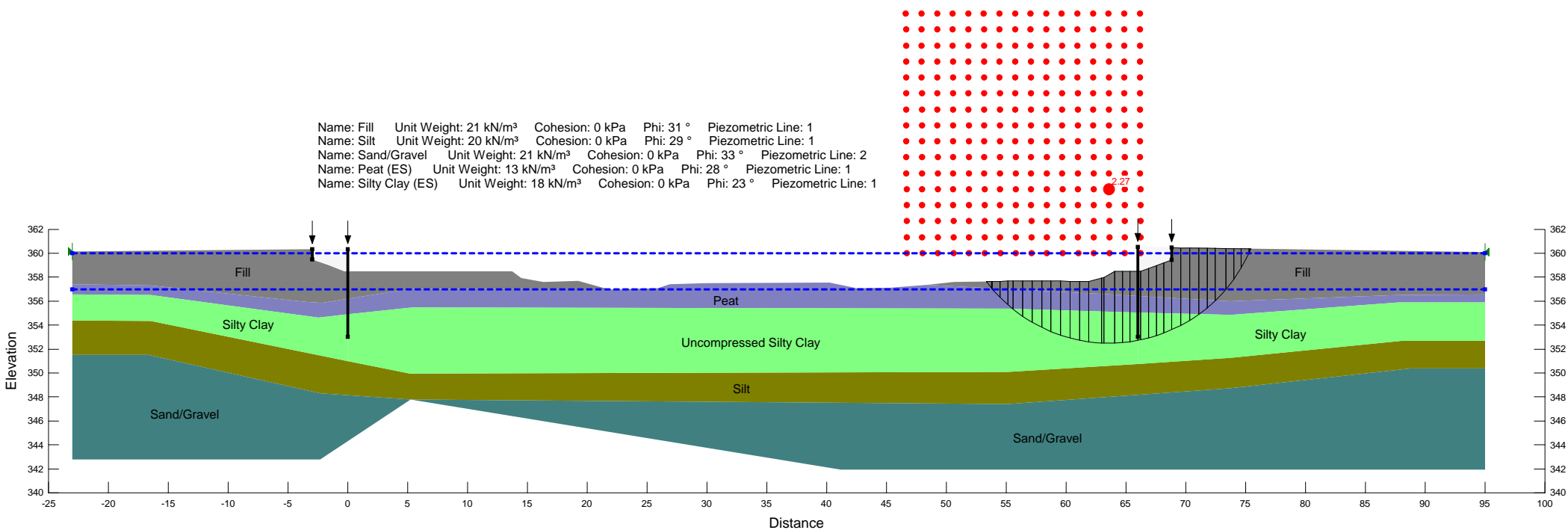
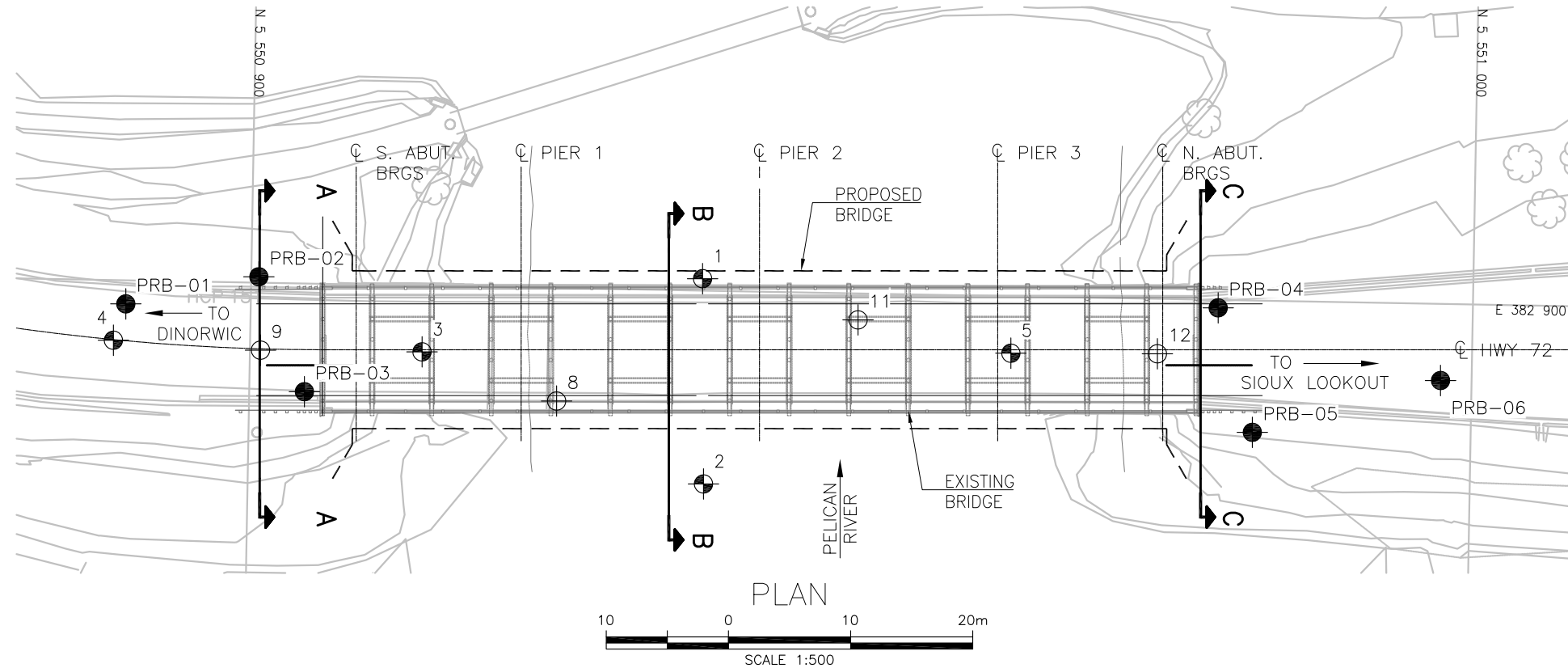


FIGURE 4

Appendix H

Borehole Locations and Soil Strata Drawings



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

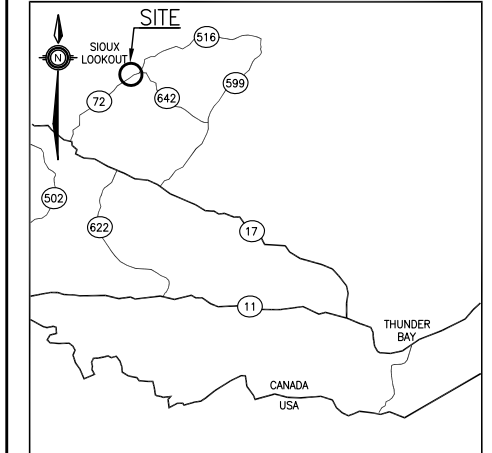


CONT No 2014-6028
WP No 6940-10-01

HIGHWAY 72
PELICAN RIVER BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET
16



KEYPLAN

LEGEND

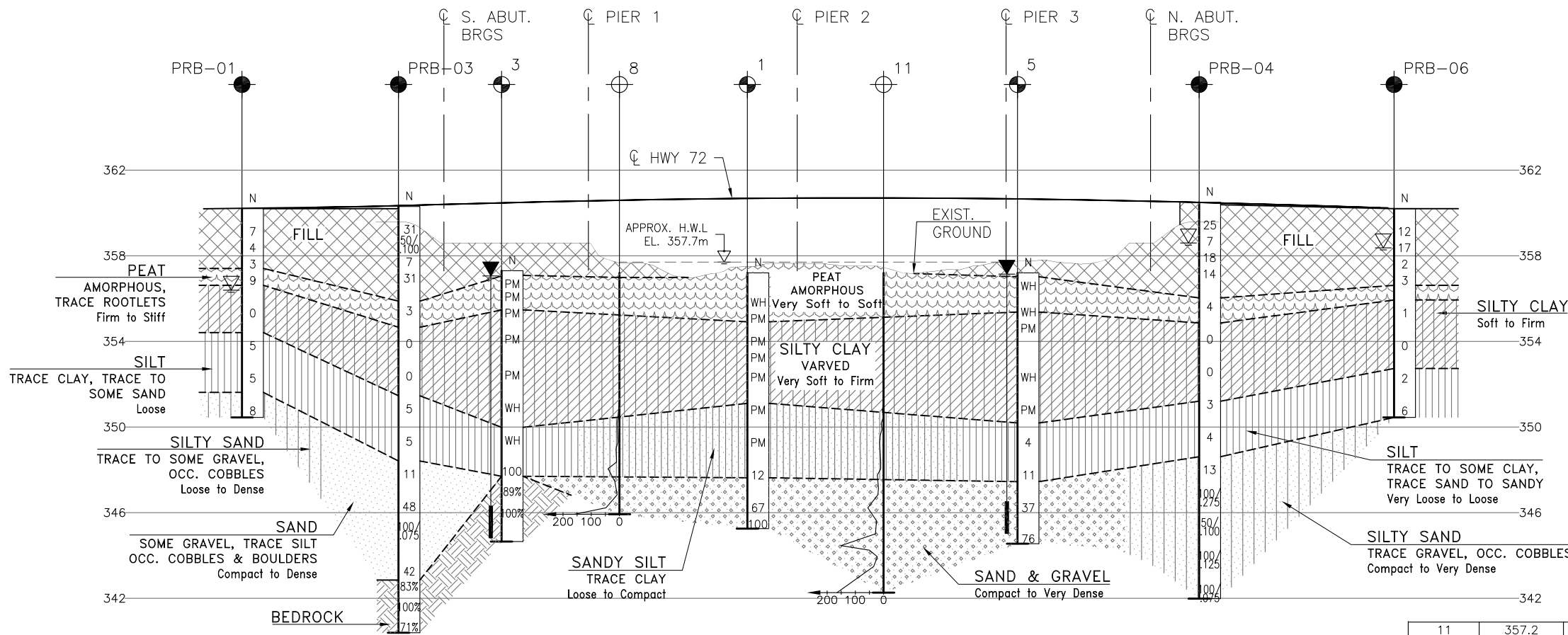
- Borehole By Thurber
- Borehole By Golder (Approx. Location)
- DCPT By Golder (Approx. Location)
- Blows /0.3m (Std Pen Test, 475J/blow)
- N Weight of Hammer
- PM Pressure, Manual
- Water Level During Drilling
- Head Level In Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
PRB-01	360.2	5 550 889.8	382 901.9
PRB-02	360.3	5 550 900.6	382 899.5
PRB-03	360.3	5 550 904.5	382 908.8
PRB-04	360.5	5 550 979.2	382 900.7
PRB-05	360.5	5 550 982.1	382 910.8
PRB-06	360.2	5 550 997.5	382 906.3
1	357.2	5 550 936.9	382 899.0
2	357.2	5 550 937.3	382 915.8
3	357.3	5 550 914.1	382 905.4
4	357.6	5 550 888.8	382 904.9
5	357.2	5 550 962.3	382 904.7
8	357.2	5 550 925.2	382 909.2
9	357.6	5 550 900.9	382 905.5

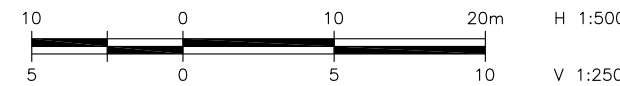
NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52J-10



PROFILE ALONG C HWY. 72



11	357.2	5 550 949.7	382 902.2
12	357.7	5 550 974.3	382 904.5

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KS	CHK	MRA
LOAD			
DATE	OCT 2014		
DRAWN	AN	CHK	KS
SITE	41S-9	STRUCT	DWG 2

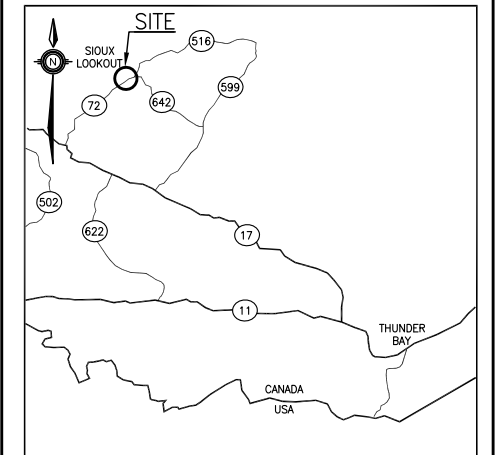
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2014-6028
WP No 6940-10-01

HIGHWAY 72
PELICAN RIVER BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
17



KEYPLAN

LEGEND

- Borehole By Thurber
- Borehole By Golder (Approx. Location)
- DCPT By Golder (Approx. Location)
- Blows /0.3m (Std Pen Test, 475J/blow)
- N
- WH
- PM
- Water Level During Drilling
- Head Level In Piezometer
- 90%
- Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
PRB-01	360.2	5 550 889.8	382 901.9
PRB-02	360.3	5 550 900.6	382 899.5
PRB-03	360.3	5 550 904.5	382 908.8
PRB-04	360.5	5 550 979.2	382 900.7
PRB-05	360.5	5 550 982.1	382 910.8
PRB-06	360.2	5 550 997.5	382 906.3
1	357.2	5 550 936.9	382 899.0
2	357.2	5 550 937.3	382 915.8
3	357.3	5 550 914.1	382 905.4
4	357.6	5 550 888.8	382 904.9
5	357.2	5 550 962.3	382 904.7
8	357.2	5 550 925.2	382 909.2
9	357.6	5 550 900.9	382 905.5

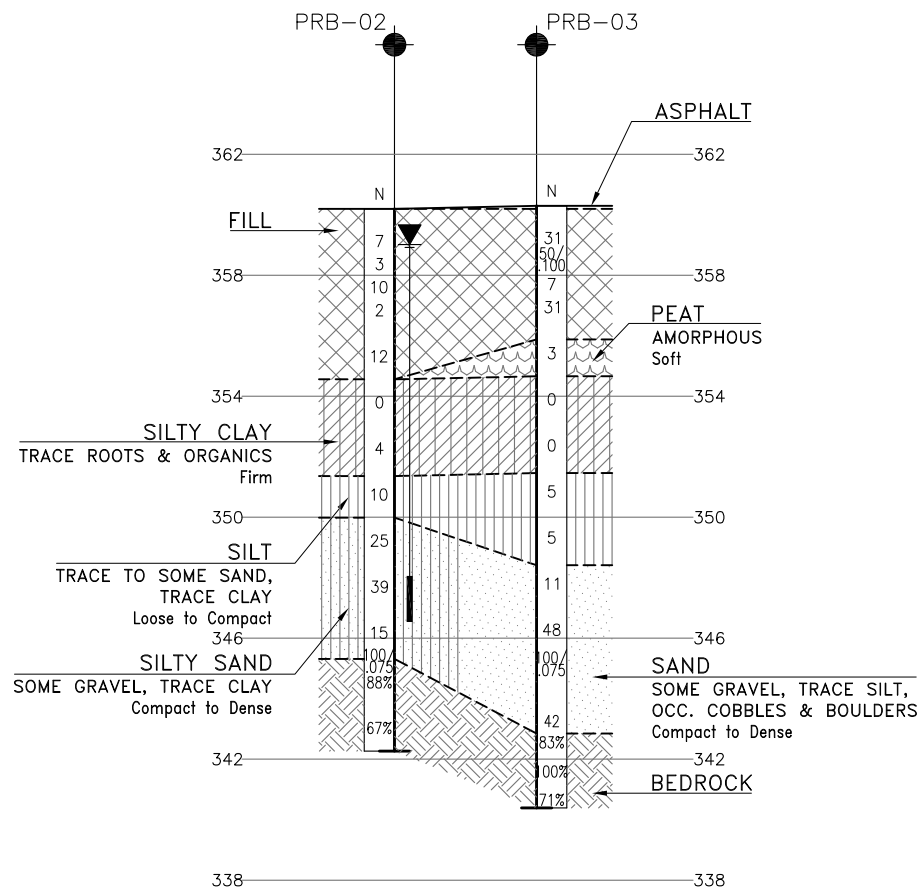
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

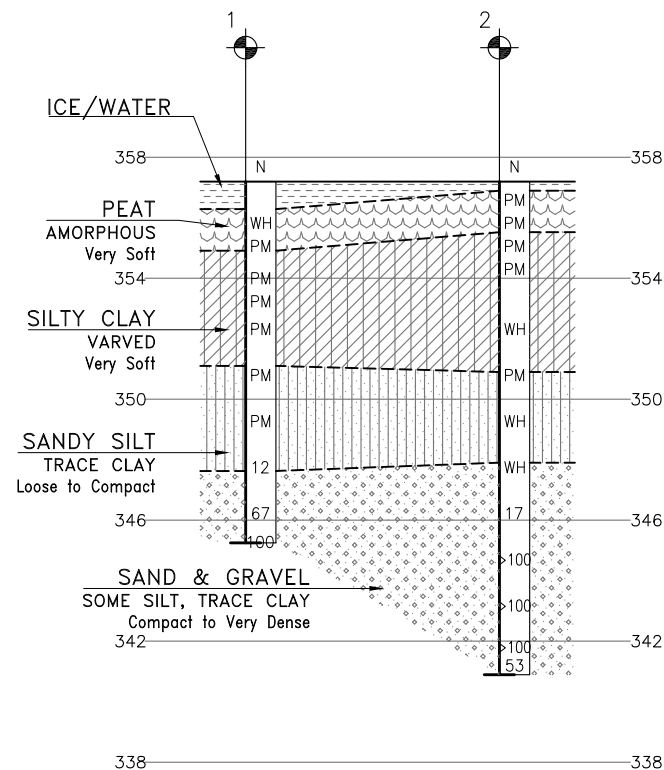
GEOCRES No. 52J-10

11	357.2	5 550 949.7	382 902.2
12	357.7	5 550 974.3	382 904.5

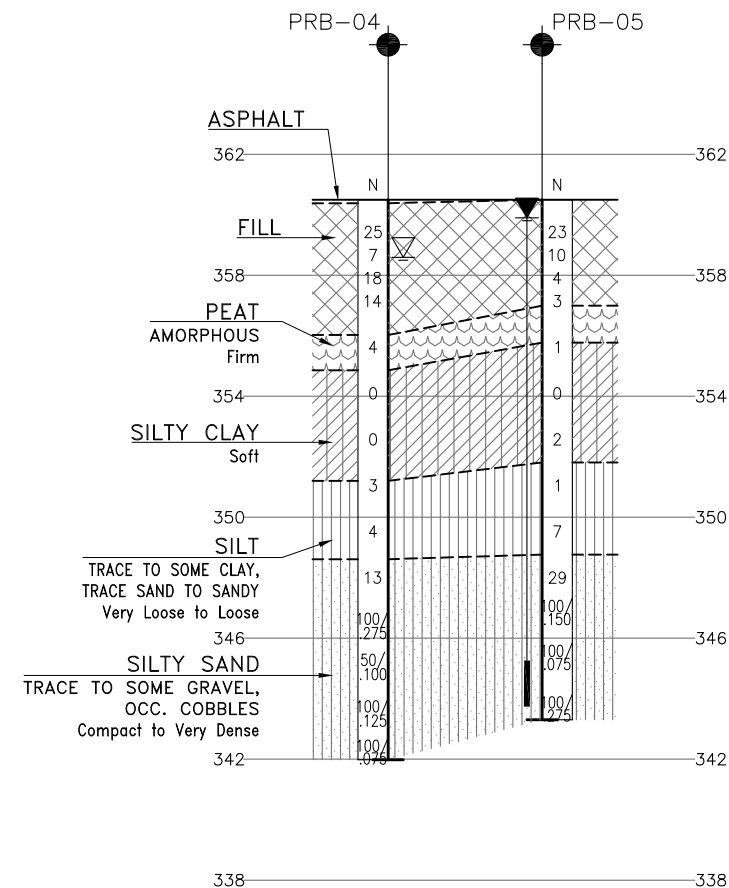
REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KS	CHK	MRA
DRAWN	AN	CHK	KS
SITE	41S-9	STRUCT	DWG 3
DATE	OCT 2014		



SECTION A-A



SECTION B-B



SECTION C-C

